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# THE SECONDARY EFFECT OF CERTAIN IMPORTANT RIVER BRIDGES ON LOCAL TRANSIT CONDITIONS

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To Be Presented May 7, 1924

## Synopsis

This paper is a study of the effect of the construction of important vehicular bridges on transit conditions in the adjacent communities. Although the primary object of building such structures is usually the accommodation of vehicular traffic, railway tracks are likely to be included, and the increased transportation facilities thus provided cause far-reaching changes in the trend of passenger traffic. In particular, the effect has been studied of the construction of the Brooklyn and Williamsburg Bridges on travel between the Boroughs of Manhattan and Brooklyn of the City of New York. As an example of the possible application of the conclusions drawn from that investigation, the situation created by the erection of the new Delaware River Bridge between Philadelphia, Pa., and Camden, N. J., has been considered. An analysis of traffic movement across the East River, New York, indicates that the existing transportation lines in New Jersey should be extended across the new bridge into Philadelphia.

## Introduction

Influenced by the great increase in the use of motor vehicles during the past few years, there has been a correspondingly greater demand for vehicular bridges. Additional facilities have been provided in many places where existing structures were inadequate, and new bridges have been built where none previously existed. To-day, there are probably more large bridges under construction or under consideration than ever before.

After an important vehicular bridge has been built across a wide river, and the communities on the two banks are thus brought more closely together, great changes may be expected in the trend of traffic. Frequently, besides the primary effect on vehicular traffic, there is also a secondary effect on local passenger transportation. In any given case, a study of the development and outcome of similar situations elsewhere, should forecast the effect that a new bridge will have on existing facilities.

An excellent example of the tremendous changes in passenger traffic which may follow the completion of an important bridge is furnished by the history of the Brooklyn Bridge. Before its construction, the only method of travel

Note.—This paper is issued before the date set for presentation and discussion. Correspondence is invited and may be sent by mail to the Secretary. Discussion on the paper will be closed in **August, 1924**, and, when finally closed, the paper, with discussion in full, will be published in *Transactions*.

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between Lower Manhattan and Brooklyn was by the East River ferries; the building of this, the first, bridge had a far-reaching effect on all local traffic, particularly that by ferry. This effect may reasonably be expected in any similar case; in fact, the construction of the Williamsburg Bridge, replacing ferry service between sections of Manhattan and Brooklyn not accommodated by the first bridge, had an almost identical effect.

However, since conditions are seldom exactly alike in any two cases, the conclusions reached from a study of East River traffic by bridge and ferry may have to be modified before estimating future traffic elsewhere.

### The Brooklyn Bridge

Prior to the opening of the Brooklyn Bridge, the traffic between Lower Manhattan and Brooklyn was carried chiefly by the five lines of the Union Ferry Company. Beginning with the most northerly, these lines were, in order: (1) Catherine Ferry, from Main Street, Brooklyn, to Catherine Street, New York; (2) Fulton Ferry, from Fulton Street, Brooklyn, to Fulton Street, New York; (3) Wall Street Ferry, connecting Montague Street, Brooklyn, and Wall Street, New York; (4) South Ferry, from Atlantic Avenue, Brooklyn, to the foot of Whitehall Street, New York; and (5) Hamilton Ferry, from Hamilton Avenue, Brooklyn, to Whitehall Street. These routes are shown in Fig. 1.

*Passenger Traffic.*—About one-half the total passenger traffic was carried by Fulton Ferry, using four boats on a 5-min. headway. The remainder was divided about equally among the other four lines, each operating on a 10-min. headway. The ferry fare was the same on all lines—1 cent during rush hours, otherwise 2 cents. Records show that the business done by the Union Ferry Company in 1871 amounted to 41 000 000 passengers, increasing by 1883 to more than 51 000 000, as indicated in Fig. 2.

On May 24, 1883, the Brooklyn Bridge was opened, and in September of that year, the operation of the Bridge Railroad (cable) was commenced. This railroad was simply a shuttle-line across the bridge, running 2-car trains and charging a 5-cent fare. As the Bridge Railroad was less convenient than the ferries, and the fare was higher, it did not at first take away much traffic from them. Approximately, 1 000 000 passengers were carried across the bridge in 1883, during the 3 months of operation; 8 500 000 in 1884; 17 000 000 in 1885; and 24 000 000 in 1886. At the same time, the pedestrian traffic increased to about 2 000 000 per year. The surprising feature of the situation, however, is that in spite of the fact that 26 000 000 people used the bridge in 1886, the ferry traffic remained at 50 000 000, or nearly as much as before the bridge was opened.

In 1886, the fare on the Bridge Railroad was reduced from 5 to 3 cents, and the trains were lengthened to three cars. This reduction made the differential between bridge and ferry rates during the greater part of the day only 2 cents on a round trip instead of 6 cents. Another factor affecting traffic in the following two years—1887 and 1888—was the general business depression. The combination of these two influences resulted in a serious loss of business to the ferries. In 1889, the Union Ferry Company carried only 35 000 000 passengers,

a loss of more than 30% from the high peak before the bridge was opened; the Bridge Railroad, on the contrary, continued to carry a heavier traffic each year, and, in 1890, actually exceeded the ferries in number of passengers, the figures being about 38 000 000 and 35 000 000, or a total of 75 000 000, including pedestrians, between Lower New York and Brooklyn. This represents an increase of 56% during the decade, as against an increase in the population of Brooklyn of only 40 per cent. It is evident, therefore, that after seven years of operation, the Brooklyn Bridge had not only robbed the ferries of a considerable part of their business, but had also created an absolutely new riding habit.

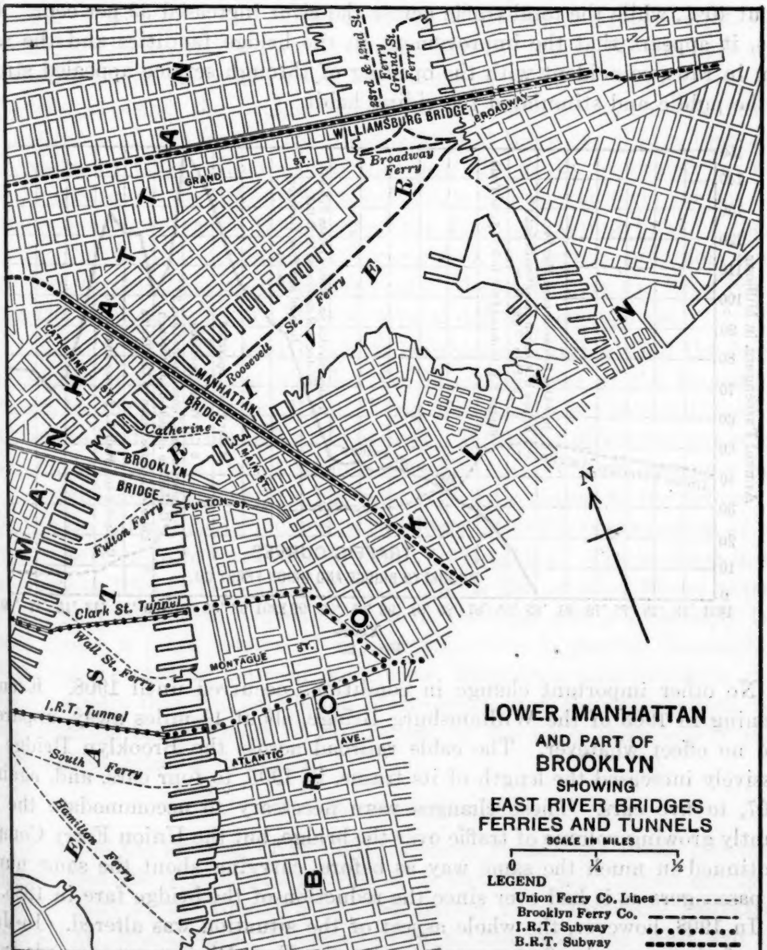


FIG. 1.

During the next ten years, the ferry traffic remained surprisingly uniform, being in 1900 only slightly less than in 1890. Meanwhile, the bridge traffic suffered a slight loss on account of the panic of 1893, but recovered soon after-

ward. On February 16, 1898, trolley service was inaugurated, making possible a continuous trip from Park Row, New York, to various points in Brooklyn. In the same year, the operation of the Bridge Railroad was taken over by the Brooklyn elevated system, and free transfers became effective between the two lines; this combination caused a tremendous jump in the bridge traffic, which practically doubled in the succeeding two years. Strangely enough, however, the ferries suffered no loss on this account; in fact, they carried more passengers in 1899 than in 1895 before the inception of trolley service on the bridge. A summary of the decade from 1890 to 1900 is almost identical with that of the preceding decade, as the population of Brooklyn again increased about 40%, while the total traffic across the river increased 59 per cent. From this, it appears that the improvement in the bridge facilities and the reduction in rates coincident with the opening of through service appealed strongly to the public and stimulated the riding habit.

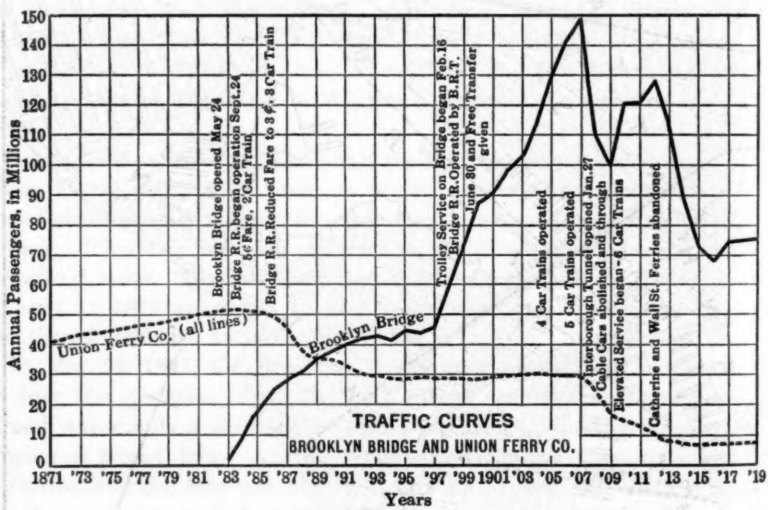

FIG. 2.

No other important change in conditions occurred until 1908. Even the opening in 1903 of the Williamsburg Bridge, about $1\frac{1}{4}$ miles away, apparently had no effect whatever. The cable railroad across the Brooklyn Bridge successively increased the length of its trains, in 1905, to four cars, and, again, in 1907, to five cars. These changes were necessary to accommodate the constantly growing volume of traffic over the bridge, but the Union Ferry Company continued in much the same way as before, carrying about the same number of passengers as it had ever since the reduction of the bridge fare in 1886.

In 1908, however, the whole aspect of the situation was altered. In January of that year the first tunnels under the East River were opened; in the same year, the operation of cable cars on the Brooklyn Bridge was abandoned and through elevated service was inaugurated, using 6-car trains running directly to the New York Terminal. In spite of better transportation on the bridge, however, the traffic diminished sharply from more than 150 000 000 in

1907 to about 110 000 000 in 1908, due undoubtedly to the opening of the subway tunnels. This also was the determining factor in the utter ruin of the passenger business of the ferries. The traffic of the Union Ferry Company was running at approximately 30 000 000 per year in 1907. Then came the slump: 1908 showed a loss of 27% from the previous year and 1909 was 32% below 1908. As a result of the opening of the Manhattan Bridge in 1910, the Catherine Street and Wall Street lines were abandoned.

*Vehicular Traffic.*—Considering briefly the question of vehicular traffic, the bridges have never appealed strongly to horse-drawn vehicles. When the first bridge was built, and for many years thereafter, all vehicular traffic was horse-drawn. The high point of the roadway of Brooklyn Bridge is 135 ft. above mean high water, and in spite of the fact that the approaches run back to points elevated above the water-front, nevertheless, a hard climb is required of vehicles crossing the bridge. With the increase of automobile traffic, the importance of this feature has diminished, but there has always been sufficient horse-drawn traffic either originating near the docks, or preferring the easy ferry passage to the difficult bridge climb, to sustain the ferry service. It was this horse-drawn traffic that enabled the three remaining lines of the Union Ferry Company to continue until recently a moderately profitable operation.

*Conclusions Drawn.*—Consideration of the traffic over the Brooklyn Bridge and the East River ferries leads to several definite conclusions:

1.—As long as the bridge fare is considerably higher than the ferry fare, passenger traffic will not abandon the latter in favor of the former; this is shown by the fact that the Union Ferry Company lost practically no business while the bridge fare was 3 cents greater. When the difference was reduced to 1 cent, however, the ferries immediately lost 30% of their passenger business.

2.—Bridge shuttle service has relatively little appeal to the public. Traffic on the Brooklyn Bridge doubled in two years, after the inauguration of through trolley service from Park Row to points in Brooklyn. The increase in the two years from 1898 to 1900 was as great as during the previous fifteen years.

3.—A bridge with convenient transit facilities and a reasonably low fare will create a large volume of new traffic. This is proved inasmuch as the total number riding between down-town Manhattan and Brooklyn increased more rapidly than the population of Brooklyn over the same period of years, the increase being wholly on the bridge, as the ferry traffic either remained constant or actually decreased.

### The Williamsburg Bridge

Prior to the opening of the Williamsburg Bridge, the traffic between Manhattan and the Williamsburg Section of Brooklyn was carried chiefly by the several lines of the Brooklyn and Manhattan Ferry Company, operated from Broadway, Brooklyn, to Roosevelt, Grand, East 23d, and East 42d Streets, Manhattan; from Grand Street, Brooklyn, to Grand Street, Manhattan; and from Greenpoint to East 10th and East 23d Streets, Manhattan. These routes provided the only transportation between the Williamsburg Section and that part of Manhattan directly across the river. The Brooklyn Bridge and the Union Ferry Company being 1¼ miles away, had no influence on the situation.

A study of passenger traffic on the Brooklyn and Manhattan ferries substantiates the conclusions already reached in connection with the Union Ferry Company.

On December 19, 1903, the roadway of the Williamsburg Bridge was opened. Nearly a year later, November, 1904, trolley service was inaugurated (a local line across the bridge only); in February, 1905, through trolley operation was started; and in the fall of 1908, rapid transit service began. To quote from Vol. 2 of the Annual Report of the Public Service Commission for the First District of New York for 1916:

"The following are the most immediate effects * * * of the opening of * * * the Williamsburg Bridge in 1904: Discontinuance in 1908 of the following ferries: Grand Street, Manhattan, to Broadway and to Grand Street, Brooklyn; Roosevelt Street, 23d Street, and 42d Street, Manhattan, to Broadway, Brooklyn."

It should be noted that the ferry lines were abandoned in 1908, the year that subway service across the bridge was inaugurated and four years after the

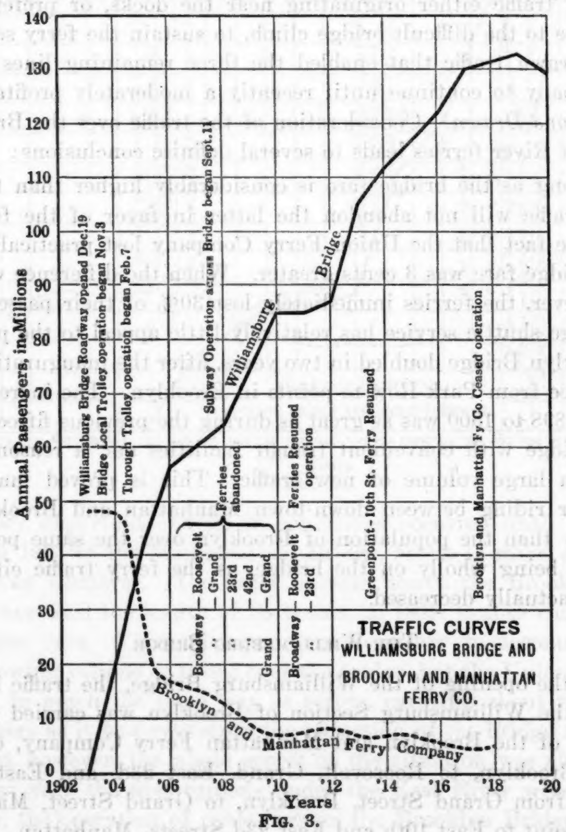


Fig. 3.

completion of the structure. Actual figures for individual ferries are not readily available. The two routes from Greenpoint, the traffic of which is included in the statistics, do not belong, perhaps, in the same category as the

others in regard to bridge competition, but because the major part of the traffic was carried on the lines operating from Grand Street and from Broadway, Brooklyn, this inclusion does not alter the situation. In 1903, the Brooklyn and Manhattan Ferry Company was carrying 48 000 000 passengers per year, or nearly as many as the Union Ferry Company in the heyday of its prosperity. In 1904, the traffic remained constant, because no trolleys were used on the bridge until November. In 1906, after through trolley service had commenced, the ferry traffic decreased to 16 000 000, a loss of 67 per cent. The year that subway service began, the ferry traffic was 14 000 000, but the year following, it was only 9 000 000. These variations are shown in Fig. 3.

Thus, the history of the Brooklyn Bridge and the Union Ferry Company was repeated without any important variation when the Williamsburg Bridge was opened—the through service across the bridge was the factor that finally spelled the doom of the ferries. It would appear that similar developments are likely to result from like conditions, wherever they occur. As an example, it may be interesting to consider the bridge now being built across the Delaware River from Philadelphia to Camden. This situation is so similar to the others that it should be possible to forecast the effect of the construction of the bridge on the local transit situation.

## The Delaware River Bridge

At present, four ferry routes cross the Delaware River between Camden and Philadelphia (Fig. 4): (1) from Cooper's Point to Shackamaxon Street, Philadelphia; (2) from Cooper's Point to Vine Street, Philadelphia; (3) from Market Street, Camden, to Market Street, Philadelphia; and (4) from Kaighn Avenue, Camden, to the foot of Chestnut Street. According to the report of the Board of Engineers to the Delaware River Bridge Joint Commission, of June 9, 1921, passenger traffic is divided among these routes, as follows: Schackamaxon Street, 1%; Vine Street, 2%; Market Street, 77%; and Kaighn Avenue, 20 per cent. The first two routes are unimportant; likewise, the most southerly, Kaighn Avenue Ferry, need hardly enter into the calculations as its traffic is largely composed of passengers of the Philadelphia and Reading Railroad, who will patronize that ferry in any case, and a few others who use it for convenience alone. It has already been shown from the history of the Brooklyn and Williamsburg Bridges that traffic crossings 1 mile or more apart are virtually independent of each other. Hence, the use of the Kaighn Avenue Ferry is unlikely to be influenced by the new bridge.

Ferry statistics for 1920 show that 37 000 000 passengers use the Market Street route, of which the Pennsylvania Railroad Company estimates that one-third uses the trains to and from the Camden Station. It appears unlikely that these travelers will be diverted to the new bridge as long as the Camden Station remains where it is. Moreover, the ferry is free to railroad passengers, and use of the bridge, therefore, would involve increased expense. Nor is it to be reasonably expected that the commuters from suburban points in New Jersey will desert the railroad in favor of other means of transportation, simply because the latter crosses the bridge into Philadelphia. The corollary of the proposition that the Pennsylvania Railroad Company will retain its present

traffic, is that Market Street Ferry will continue operation indefinitely, although possibly with less frequent service. A certain amount of horse-drawn traffic will probably continue to patronize the ferry, as in the case of the Union Ferry Company, because, at Philadelphia, many of the produce warehouses are located near the water-front.


FIG. 4.

One-third of the total traffic of this ferry consists of through railroad passengers who have no interest in local transportation in Camden and vicinity; the remaining two-thirds, numbering about 24 600 000 per year, or 67 500 per day, use trolley or busses, or walk to near-by points. The effect of the new Delaware River Bridge on the local transit situation resolves itself into a question of what these daily local passengers will do. Referring to the history of the East River bridges in this connection, it is important to note some of the

differences between the two situations. For instance, the ultimate destination of "Brooklynites" bound to Lower Manhattan is usually within easy walking distance of ferry and bridge terminals, whereas the ultimate destination of "Camdenites" bound to Philadelphia is apt to be some distance away from the ferry terminal at the foot of Market Street. Again, when the Brooklyn Bridge was built, the population of that Borough was several times as large as the present population of Camden.

In the report of the Board of Engineers to the Delaware River Bridge Joint Commission, previously mentioned, it is proposed that there shall be four tracks on the bridge, two for interurban rapid transit trains, and two for a trolley shuttle-line. It is further suggested that there shall be constructed at some indefinite date in the future an elevated rapid transit system in New Jersey which shall use the bridge and the existing subway tracks of the Philadelphia Rapid Transit Company, thus connecting various suburban points directly with the heart of Philadelphia. As an ultimate solution of the transportation question in this area, the plan is not without attractive features, but as a possibility for the near future it is hardly thinkable—the cost of construction would be out of all proportion to the present traffic demands of Camden and its environs.

There are, however, several ways of utilizing the tracks, without involving expensive new elevated construction work:

1.—A shuttle-line, as proposed in the report to the Joint Commission, previously mentioned, crossing the bridge and looping in Camden *via* Fourth, Mickle, and Fifth Streets. Existing transportation agencies would continue operation substantially as at present.

2.—Operation across the bridge of either the Philadelphia subway or surface lines to a terminus at the Camden end.

3.—Operation of the Camden lines across the bridge to a terminus at the Philadelphia end.

4.—Some combination of the foregoing plans.

*Shuttle Service.*—In case shuttle service is operated, the length of the route will be about 4 miles per round trip. A fairly short headway would always be necessary, because people are used to frequent ferry service; an even shorter headway would be required to take care of the sharp rush-hour peaks. On a 4-min. schedule, this would mean at least 300 trips per day, or 1 200 car-miles. The annual operating expense, at 40 cents per car-mile, would be $168 000.

It is difficult to estimate the exact number of passengers who would prefer such service to the present ferries. Although the shuttle had little effect on the East River ferries, nevertheless the fact that the bridge terminal is more centrally located than the ferry terminal in Philadelphia might make such service slightly more attractive to "Camdenites" than it proved to be to Brooklynites". New Jersey people desiring to reach a point within easy walking distance of the Pennsylvania end of the bridge would probably save money by paying one fare on the bridge shuttle instead of paying 4 cents on the ferry and 7 cents in Philadelphia. On the other hand, those whose destination is more or less remote from the bridge terminal, would still have to

take local transportation in Philadelphia, and unless the bridge fare was less than 5 cents, they would be paying more money by using the shuttle. Moreover, for a person who uses local transportation in both cities, a short ferry ride is a pleasant interlude, and the substitution of another rail trip for his present water trip would have little appeal.

The assumption that the traffic on a shuttle-line would approximate the number of people who now walk to and from the ferries in Camden, would probably not be far wrong. From Fig. 5, the total number of pedestrians (both directions) is shown to be about 15 000, of which number, 6 000 are carried by the ferries in the two rush hours—eastward in the morning and westward at night. On a basis of 100 passengers per car, the 4-min. headway of the shuttle cars would accommodate 1 500 per hour as a maximum, and thus reduce the possible total passengers to 12 000 per day. The present local fare on the Williamsburg Bridge is $1\frac{2}{3}$ cents, on the Brooklyn Bridge, $2\frac{1}{2}$ cents, and on the Market Street Ferry, 4 cents. In view of the conclusions reached by a study of the traffic on those two bridges, the maximum fare possible on the Delaware River Bridge shuttle appears to be 3 cents. The annual revenue would then be \$126 000, or \$42 000 less than the estimated costs. It would be impossible to justify a larger estimate of traffic on a bridge shuttle-line on the ground of experience elsewhere. Under these circumstances, the plan can be dismissed as impractical, because it would be neither popular or profitable.

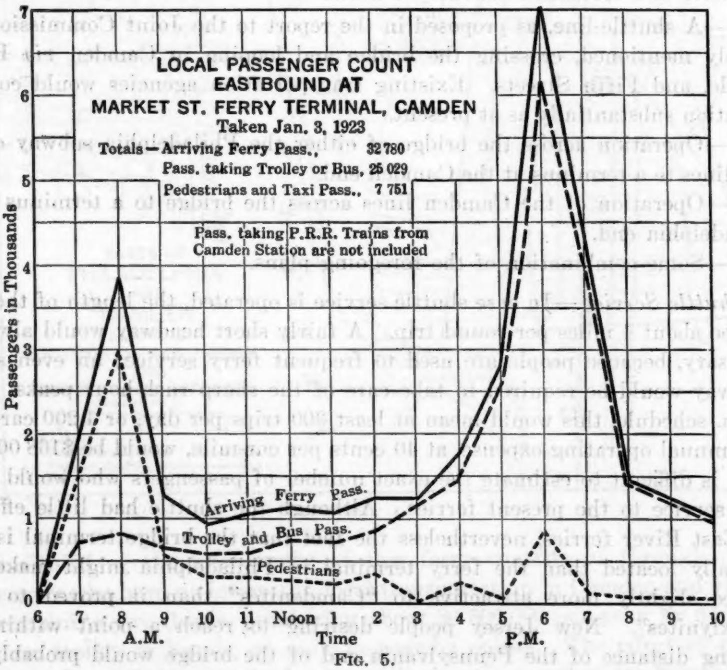


FIG. 5.

*Extension of Philadelphia Lines.*—The next possibility to be considered is the operation of the subway trains of the Philadelphia Rapid Transit Company across the bridge to a terminus in Camden. This seems only slightly less

improbable than the plan already discussed. The bridge terminal in Philadelphia will be so far back from the river and so far north of the present subway under Market Street, that it would be no easy matter to get the trains to the bridge. Moreover, in so doing, subway service to the lower end of Market Street and to the ferries would be sacrificed. Much additional mileage would be added to the operation if Camden was made the terminus in place of the Philadelphia water-front. It would hardly be feasible to split the service, sending some across the new bridge and some to the ferries, because it is already split between Frankford and the ferries. As the ultimate destination in Philadelphia of ferry passengers is probably the vicinity of the City Hall, the majority of them now use the Philadelphia Rapid Transit lines. There would, therefore, be no increase in traffic to this Company if it were to operate over the bridge. Unless an extra fare across the bridge should be charged, there would be no increase in revenue to counterbalance the added mileage and the traffic lost by failure to serve Lower Market Street.

Similar criticisms can be made of projects to send Philadelphia trolleys across to Camden. Although the latter could be arranged with less construction cost, the same loss of traffic on parts of their present routes and the same increased mileage without increased revenue would result. From every point of view, it is undesirable to tie up the Philadelphia transportation system, the primary mission of which is to serve that city, with bridge service of an entirely different nature.

*Use of Bridge by Camden Trolleys.*—Operation over the Delaware River Bridge from Camden to Philadelphia, on the other hand, is a more attractive plan. Of the 65 000 daily local passengers on the Market Street Ferry, 76% now use the trolleys or busses in Camden and approximately 24% walk (Fig. 5). With cars running through from the New Jersey side directly into Philadelphia, a large proportion of the pedestrians probably would become passengers, because the inevitable curtailment of ferry service would make the bridge the natural route for crossing the river. Although not all of them would necessarily ride across the bridge, nevertheless, it has been the experience on the Brooklyn and Williamsburg Bridges that the number of pedestrians is inconsequential compared to the rail passengers, averaging less than 3% of the total. Assuming that one-half the people who now walk to the ferry, would take transportation across the bridge if no extra fare was charged, this would give a traffic increase of 8 000 per day, or 2 800 000 per year. It will be seen that this figure is smaller than the traffic estimate already given for a bridge shuttle-line. This is natural if the existing transportation agency charges its regular fare, which is higher than that which could reasonably be asked on a shuttle-line. Hence, fewer pedestrians would become riders. To counterbalance that, however, is the fact that it would cost much less for the present lines to extend their service across the new bridge, than for a shuttle-line to operate thereon. The volume of travel between Philadelphia and Camden has been increasing steadily, as shown in Fig. 6, and with the completion of the bridge may be expected to increase even more rapidly. This increase in the riding habit would soon greatly augment the number of passengers, and even

after allowing for rather wide variation in these figures, the plan remains a promising one.

*Possible Combined Plans.*—The fourth alternative is a combination of two of the plans already mentioned. It is evident that the shuttle service would be unprofitable even though operated as a monopoly on the bridge; if operated alongside of through service, it would be even more unprofitable. The disadvantages incident to the operation of the Philadelphia lines by themselves across the bridge weigh equally against any combination plan.

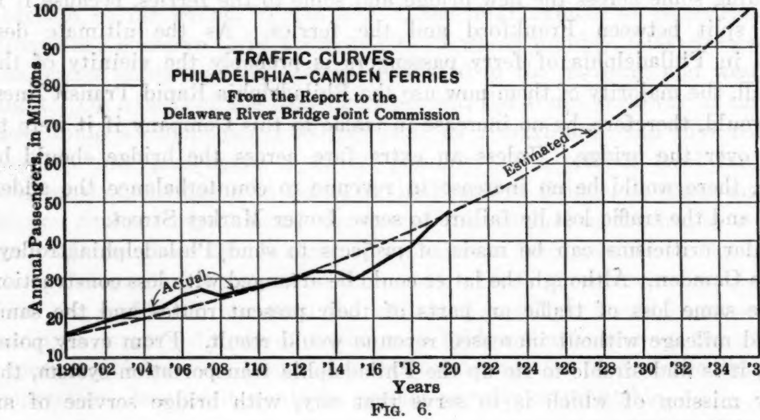


FIG. 6.

The fact that operation across the Delaware River Bridge would involve interstate transportation is an important consideration. Probably neither railway is desirous of coming under the jurisdiction of the Interstate Commerce Commission, and a profitable business would have to be assured before either company would undertake such operation. Then, too, the present population of Camden and its environs is much smaller than that of Brooklyn in 1883, so that no such volume of traffic as that carried by the Brooklyn Bridge in its early days could be expected for many years. On the other hand, one of the existing transportation agencies, rather than see its territory invaded by another. might prefer to give a service even at a sacrifice, in order to keep out competitors.

*Comparison of Plans.*—In view of the history of the East River bridges, the most logical program appears to be, therefore, that the local transportation lines in New Jersey extend their service across the new Delaware River Bridge to Philadelphia. Such a development would be in harmony with the true mission of these lines, which is to bring New Jersey suburbanites into the city. Moreover, it presents no serious construction problems, and is economically the most promising of the four plans.

## CONCLUSION

Similar bridges are being considered elsewhere, as, for example, from New York, across the Hudson River, to New Jersey. In such cases, the general conclusions reached from a study of the East River bridges would probably be applicable, and should receive careful consideration before the adoption of final plans for the rail service involved.

# ANALYTICAL SOLUTION OF MASONRY DOMES

By David C. Coyle,* Esq.

## Synopsis

The object of this investigation was to develop analytical formulas for pyramoidal and conoidal masonry domes, analogous to the equations which have been established for spherical and conical shapes. The usual method of considering the forces on an elementary voussoir was adopted; that is, all the factors which may vary independently were kept separate, and appear in the final equations. These equations give the meridional and ring stresses at any point in any dome the plan of which is radially symmetrical. There is also an expression for the strength required in the tension ring at the top of a cylindrical support for a dome. The formulas have this advantage, that the designer, once having assented to them, can comfortably forget how they were derived.

Having developed these general equations, it was necessary to justify them by showing that the established formulas for special cases can be derived from them. This proves to be true in every instance.

Finally, the general equations give occasion to discuss certain facts which have been overlooked or confused in some of the previous writings on the subject. The paper aims to show that the conclusions derived from the formulas are those which might be expected and involve no *reductio ad absurdum.*

The following discussion deals with the stresses in a dome of any profile that is symmetrical about the vertical axis. The formulas submitted are simpler than any which the writer has seen, and it will be shown that they are consistent with the established equations for spherical and conical domes.

In all cases, the line of stress is assumed to lie in the middle of the section as in most modern treatments of the subject. As domes are usually monumental, the increased factor of safety is no detriment, whereas there is the added advantage of analytical formulas which are simple enough to be of practical use. The formulas apply approximately to any line of stress whether in the center or not.

## Notation

In the following notation (Fig. 1), units are given in pounds and feet:

$A$ = meridional stress, in pounds per foot of horizontal section.
$B$ = ring stress, in pounds per foot of profile.

Note.—Written discussion on this paper, which will not be presented at any meeting of the Society, will be closed with the August, 1924, *Proceedings*. When finally closed, the paper, with discussion, will be published in *Transactions*.

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$C$ = tension in the top ring of a vertical support or compression in the bottom ring of a vertical lantern, in pounds.

$W$ = sum of all loads above a given level, $x$.

$p$ = average weight of shell per square foot at level, $x$.

$R$ = radius of curvature of profile at $x$.

$r$ = horizontal distance from $x$ to axis of dome.

$\theta$ = angle between radius, $R$, and the horizontal.

$H$ = radial bursting force on a circle of radius, $r$, for an area of 1 ft. of circumference $\times$ 1 ft. of profile, in pounds per square foot.

The following applies to pyramoidal domes:

$R$ = radius of curvature of the profile on a diagonal, at level of $x$.

$D$ = semi-diagonal of horizontal section at $x$.

$d$ = semi-diameter of horizontal section at $x$.

$n$ = number of sides of polygonal horizontal section.

$L$ = side of polygon, horizontal section at $x$.

$\phi$ = angle between diagonal and side of polygon.

$M$ = bending in a horizontal element at level, $x$, in foot-pounds per foot of profile.

The following notation applies to spherical domes:

$S_0$ = height from top of masonry to point, $x$.

$p$ = uniform weight per square foot of shell.

$h$ = height from center of sphere to point, $x$.

$V$ = weight of lantern, in pounds.

$w$ = weight per cubic foot of shell.

$t$ = thickness of shell $= \dfrac{p}{w}$

The following applies to conical domes:

$a$ = vertical height of apex of cone above $x$.

$h$ = vertical height of top of masonry above $x$.

$s$ = slant height of top of masonry above $x$.

$r_0$ = horizontal radius at top of masonry.

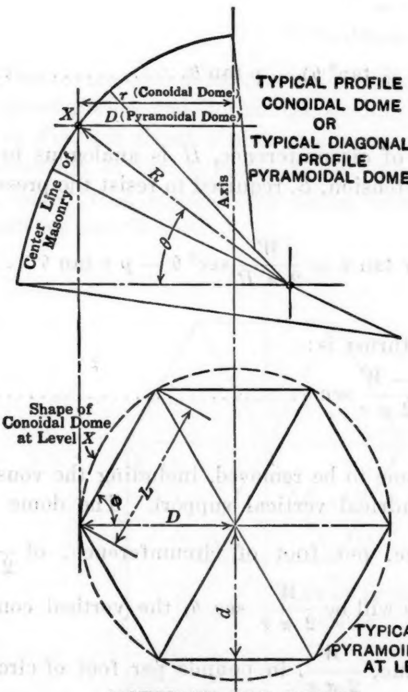
$p$ = uniform weight of shell per square foot.

### Conoidal Domes

Fig. 2 shows a vertical section of part of a conoidal dome, that is, a dome generated by revolving any profile about a vertical axis. The forces used are those acting on a voussoir, 1 ft. long and $R\,d\,\theta$ high, at any point, $x$.

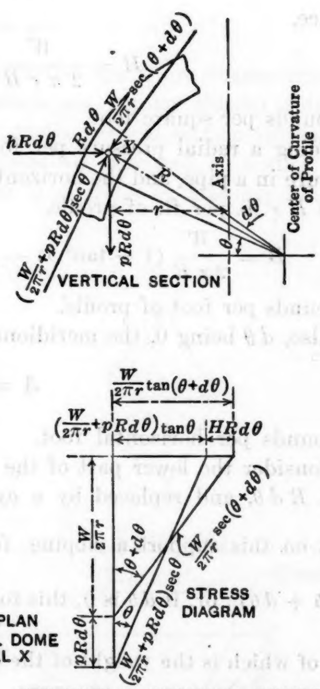
The following forces act on the voussoir: The meridional thrust from above $= \dfrac{W}{2\,\pi\,r}\sec(\theta + d\,\theta)$; the horizontal component $= \dfrac{W}{2\,\pi\,r}\tan(\theta + d\,\theta)$; and the vertical component $= \dfrac{W}{2\,\pi\,r}$. The weight of the voussoir $= p\,R\,d\,\theta$.

The meridional thrust from below (assumed to be tangent to the profile) $= \left(\frac{W}{2\,\pi\,r} + p\,R\,d\,\theta\right)$ sec $\theta$; the horizontal component $= \left(\frac{W}{2\,\pi\,r} + p\,R\,d\,\theta\right)$ tan $\theta$; and the vertical component $= \frac{W}{2\,\pi\,r} + p\,R\,d\,\theta$.




NOTATION FOR DOMES

FIG. 1.




STRESSES IN CONOIDAL DOMES

FIG. 2.

The radial force necessary to keep the thrust in the center of the shell, as assumed (sum of all the other horizontal components) $= H\,R\,d\,\theta$. This force represents the tendency of the dome to burst.

Then:

$$H\,R\,d\,\theta = \frac{W}{2\,\pi\,r}\tan(\theta + d\,\theta) - \left(\frac{W}{2\,\pi\,r} + p\,R\,d\,\theta\right)\tan\theta \ldots\ldots(1)$$

Now,

$$\tan(\theta + d\,\theta) = \frac{\tan\theta + \tan d\,\theta}{1 - \tan\theta\tan d\,\theta}$$

and,

$$H\,R\,d\,\theta = \frac{W}{2\,\pi\,r}\,\frac{\tan d\,\theta + \tan^2\theta\tan d\,\theta}{1 - \tan\theta\tan d\,\theta} - p\,R\,d\,\theta\tan\theta \ldots\ldots(2)$$

Now, let $d\,\theta$ approach 0; then,

$$\tan d\,\theta = \frac{R\,d\,\theta}{R} = d\,\theta$$

and,

$$H\,R = \frac{W}{2\,\pi\,r}\,\frac{1 + \tan^2\theta}{1 - 0} - p\,R\tan\theta$$

whence,

$$H = \frac{W}{2\,\pi\,r\,R}\,(1 + \tan^2\theta) - p\tan\theta \ldots\ldots\ldots\ldots\ldots\ldots (3)$$

in pounds per square foot.

Being a radial pressure per foot of circumference, $H$ is analogous to the pressure in a pipe, and the horizontal tension, $B$, required to resist the pressure, $H$, is $H\,r$ lb. per ft. of profile.

$$B = \frac{W}{2\,\pi\,R}\,(1 + \tan^2\theta) - p\,r\tan\theta = \frac{W}{2\,\pi\,R}\sec^2\theta - p\,r\tan\theta \ldots\ldots (4)$$

in pounds per foot of profile.

Also, $d\,\theta$ being 0, the meridional thrust is:

$$A = \frac{-W}{2\,\pi\,r}\sec\theta \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (5)$$

in pounds per horizontal foot.

Consider the lower part of the dome to be removed, including the voussoir ring, $R\,d\,\theta$, and replaced by a cylindrical vertical support. The dome will exert on this support a sloping force, per foot of circumference, of $\frac{W}{2\,\pi\,r}$ $\sec(\theta + d\,\theta)$; or, if $d\,\theta$ is 0, this force will be $\frac{W}{2\,\pi\,r}\sec\theta$, the vertical component of which is the weight of the dome, $\frac{W}{2\,\pi\,r}$, in pounds per foot of circumference, and the horizontal component of which is $\frac{W}{2\,\pi\,r}\tan\theta$, in pounds per foot of circumference.

The corresponding ring tension in the top of the cylinder is this horizontal thrust $\times\,r$, or

$$C = \frac{W}{2\,\pi}\tan\theta \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (6)$$

in pounds.

At the top of an open dome, a cylindrical load, such as a lantern, may be placed. This load requires a compression ring in the bottom, analogous to the tension ring which has just been considered.

The load being $W$, it is upheld by a series of sloping forces the value of which is $\frac{W\sec\theta}{2\,\pi\,r}$ per foot, and the vertical component of which is the load, $W$, or, $\frac{W}{2\,\pi\,r}$ per foot, and the horizontal component of which is $\frac{W}{2\,\pi\,r}\tan\theta$.

The corresponding ring compression is, therefore,

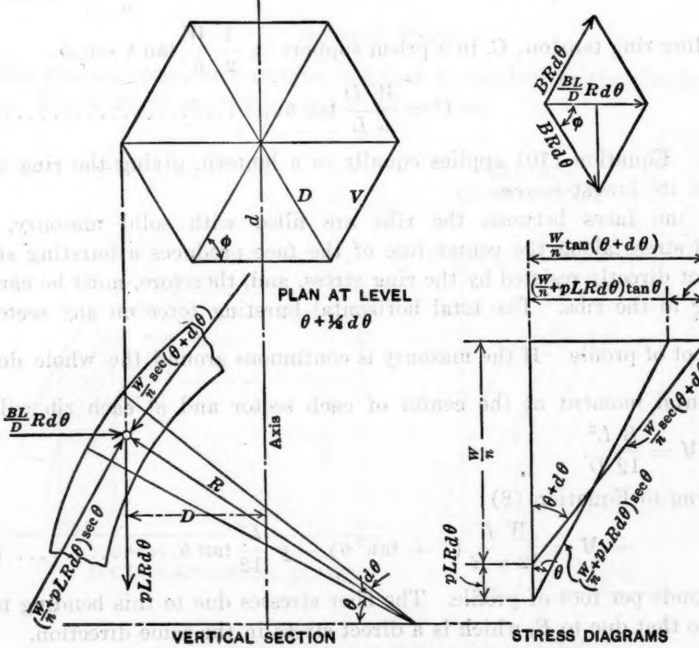
$$C = \frac{-W}{2\,\pi} \tan \theta$$

Equations (4), (5), and (6) are general for all conoidal domes. The corresponding expressions for pyramoidal domes, of which the conoid is a special case, will now be derived.

## Pyramoidal Domes

The pyramoidal domes, to which these formulas apply, are characterized by a curved profile and horizontal sections which are regular similar polygons.

In Fig. 3 is shown a plan of a pyramoid, and a vertical section through the intersection of two adjacent faces. The loads are assumed as acting along the corner ribs, whether the filling between is of glass or masonry, and are shown for one rib only.




STRESSES IN PYRAMOIDAL DOMES

Fig. 3.

If $B\,R\,d\,\theta$ is the ring tension, the radial resultant is $2\,B\,R\,d\,\theta \cos \phi$; $\cos \phi = \frac{L}{2\,D}$. The radial bursting force then $= \frac{B\,R\,L}{D}\,d\,\theta =$ the difference between the horizontal components of the meridional thrusts above and below the voussoir.

$$\frac{B\,R\,L}{D}\,d\,\theta = \frac{W}{n}\tan(\theta + d\,\theta) - \left(\frac{W}{n} + p\,L\,R\,d\,\theta\right)\tan\theta$$

$$= \frac{W}{n}\left[\frac{\tan\theta + \tan d\,\theta}{1 - \tan\theta\tan d\,\theta} - \tan\theta\right] - p\,L\,R\,d\,\theta\tan\theta \text{.........(7)}$$

As before, with $d\,\theta = \tan d\,\theta$ approaching 0,

$$\frac{B\,R\,L}{D} = \frac{W}{n}\,\frac{1 + \tan^2\theta}{1 - 0} - p\,L\,R\tan\theta$$

$$B = \frac{W\,D}{n\,L\,R}(1 + \tan^2\theta) - p\,D\tan\theta \text{..............(8)}$$

in pounds per foot of rib profile.

Similarly, as $d\,\theta$ becomes 0, the meridional stress, $A$, equals

$$\frac{W}{n}\sec\theta \text{..........................(9)}$$

in pounds per rib.

The horizontal component of the meridional thrust, $A = \frac{W}{n}\tan\theta$, and the corresponding ring tension, $C$, in a prism support $= \frac{1}{2}\,\frac{W}{n}\tan\theta\sec\phi$.

$$C = \frac{W\,D}{n\,L}\tan\theta \text{........................(10)}$$

in pounds. Equation (10) applies equally to a lantern, giving the ring compression in its lowest course.

Where the faces between the ribs are filled with solid masonry, the meridional stress along the center line of the face produces a bursting stress which is not directly resisted by the ring stress, and, therefore, must be carried in bending to the ribs. The total horizontal bursting force on any sector is $\frac{B\,L}{D}$ per foot of profile. If the masonry is continuous around the whole dome, the horizontal moment at the center of each sector and at each rib will be about, $\pm M = \frac{B\,L^2}{12\,D}$.

Referring to Equation (8):

$$\pm M = \frac{W\,L}{12\,n\,R}(1 + \tan^2\theta) - p\,\frac{L^2}{12}\tan\theta \text{..............(11)}$$

in foot-pounds per foot of profile. The fiber stresses due to this bending must be added to that due to $B$, which is a direct stress in the same direction.

In the case of the conoid, $D = d = r$; $n\,L = 2\,\pi\,D$; $L = 0$; $\frac{1}{n} = 0$. Equations (8), (9), and (10) reduce to Equations (4), (5), and (6), respectively, while Equation (11) becomes $M = 0$.

*Stresses at Crown.*—At a point near the top of any closed dome, the following conditions prevail: Area of surface of cone of base radius, $r$, = perimeter of base $\times \frac{1}{2}$ slant height $= 2\,\pi\,r\left(\frac{r}{2\sin\theta}\right) = \frac{\pi\,r^2}{\sin\theta}$; and $W = \frac{p\,\pi\,r^2}{\sin\theta}$.

$$A = \frac{-W}{2 \pi r} \sec \theta = \frac{-p\, r}{2 \sin \theta} \sec \theta \ldots\ldots\ldots\ldots\ldots\ldots (12)$$

$$B = \frac{W}{2 \pi R} \sec^2 \theta - p\, r \tan \theta = \frac{p\, r^2}{2\, R \sin \theta} \sec^2 \theta - p\, r \tan \theta \ldots\ldots (13)$$

In the general case, the center of curvature is not on the axis, the dome is pointed at the crown, $R$ is not vertical, and none of the functions of $\theta$ approaches 0 toward the top. $A$ and $B$, being direct functions of $r$, $= 0$ at the top where $r = 0$.

Where the dome is rounded at the crown, the center of curvature is on the axis, and $\theta$ is 90° at the top. Then, $\sec \theta = \frac{R}{r}$, $\tan \theta = \frac{R}{r}$, $\sin \theta = 1$.

$$A = \frac{-p\, r}{2} \frac{R}{r} = -p \frac{R}{2} \ldots\ldots\ldots\ldots\ldots\ldots (14)$$

$$B = \frac{p\, r^2}{2\, R} \frac{R^2}{r^2} - p\, r \frac{R}{r} = -p \frac{R}{2} \ldots\ldots\ldots\ldots\ldots\ldots (15)$$

In no case is there any infinite stress at the top, as some writers have surmised.

## Simpler Types

The general conoidal formulas will now be applied to the simpler types of dome. Fig. 4 shows one with a spherical surface.

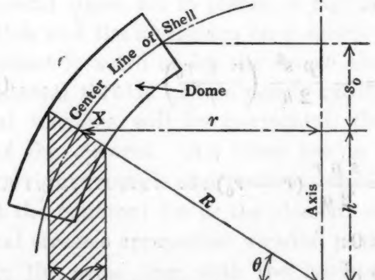



FIG. 4.

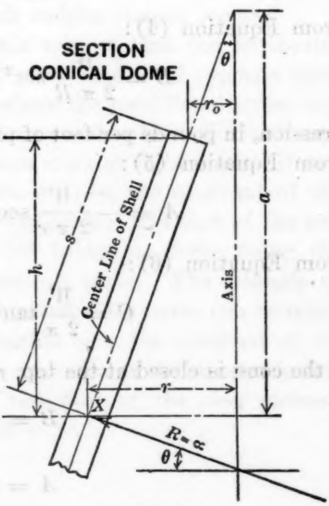



FIG. 5.

*Spherical Dome.*—The area of the surface the depth of which is $S_0 = 2 \pi R S_0$; $W = 2 \pi R S_0 p$; $\tan \theta = \frac{h}{r}$; and $\sec \theta = \frac{R}{r}$.

From Equation (4):

$$B = \frac{W}{2 \pi R} \sec^2 \theta - p\, r \tan \theta = \frac{R^2}{r^2} S_0\, p - p\, h \ldots\ldots\ldots (16)$$

From Equation (5):

$$A = -\frac{W}{2\,\pi\,r}\sec\theta = -\frac{R^2}{r^2}S_0\,p \ldots\ldots\ldots\ldots\ldots (17)$$

From Equation (6):

$$C = \frac{W}{2\,\pi}\tan\theta = \frac{R}{r}S_0\,p\,h \ldots\ldots\ldots\ldots\ldots (18)$$

Equations (16) and (17) have been published previously,* but Equation (18) is, the writer believes, original.

If a lantern weighs $V$ lb., and the surface of the dome is $t$ ft. thick and weighs $w$ lb. per cu. ft., from Equation (4):

$$B = \frac{2\,\pi\,R\,w\,t\,S_0}{2\,\pi\,R}\frac{R^2}{r^2} + \frac{V}{2\,\pi\,R}\frac{R^2}{r^2} - w\,t\,r\,\frac{h}{r}$$

$$= w\,t\,S_0\frac{R^2}{r^2} + \frac{V\,R}{2\,\pi\,r^2} - w\,t\,h \ldots\ldots\ldots\ldots\ldots (19)$$

Equation (19) was derived and used by William Cain, M. Am. Soc. C. E.†

*Conical Dome.*—Fig. 5 shows a conical dome, for which the surface $= s\,\pi\,(r + r_0)$; $W = s\,\pi\,p\,(r + r_0)$; $R =$ infinity; $\frac{1}{R} = 0$; $\tan\theta = \frac{r}{a}$; and $\sec\theta = \frac{s}{h}$.

From Equation (4):

$$B = \frac{W}{2\,\pi\,R}\sec^2\theta - p\,r\tan\theta = -\frac{p\,r^2}{a} \ldots\ldots\ldots\ldots (20)‡$$

compression, in pounds per foot of profile.

From Equation (5):

$$A = -\frac{W}{2\,\pi\,r}\sec\theta = -\frac{p\,s^2}{2\,h}\left(\frac{r + r_0}{r}\right) \ldots\ldots\ldots (21)§$$

From Equation (6):

$$C = \frac{W}{2\,\pi}\tan\theta = \frac{s\,p\,r}{2\,a}(r + r_0) \ldots\ldots\ldots\ldots (22)$$

If the cone is closed at the top, $r_0 = 0$:

$$B = -\frac{p\,r^2}{a} \ldots\ldots\ldots\ldots\ldots (23)\|$$

$$A = -\frac{p\,s^2}{2\,a} \ldots\ldots\ldots\ldots\ldots (24)¶$$

$$C = \frac{s\,p\,r^2}{2\,a} \ldots\ldots\ldots\ldots\ldots (25)$$

* *Transactions*, Am. Soc. C. E., Vol. LII (1904), pp. 269 and 312.

† *Loc. cit.*, Vol. LV (1905), p. 205, Equation (4).

‡ *Loc. cit.*, p. 225, Equation (9).

§ *Loc. cit.*, Vol. LII (1904), p. 303, Equation (73).

‖ *Loc. cit.*, Vol. LIV (1904), p. 314, Equation (71*a*).

¶ *Loc. cit.*, p. 314, Equation (70*a*).

*Cylindrical Wall.*—If the cone becomes a cylinder: $\frac{1}{a} = 0$, $h = s$, and $r = r_0$. From Equation (20), $B = 0$, that is, there is no horizontal stress, and from Equation (21), $A = -\frac{p\,h^2}{2\,h} \times \frac{2\,r}{r} = -p\,h$, which is the vertical load per foot of circumference. From Equation (22), $C = 0$, that is, no tension ring is required under the bottom of a cylinder.

## Remarks on Formulas

A number of interesting conclusions may be drawn from these equations. It is inaccurate to state that a full hemispherical dome has no tension around the bottom; Equation (4) shows that for $\tan\theta = 0$, $B = \frac{W}{2\,\pi\,R}$, which is the greatest tension anywhere in the dome. On the other hand, $C = 0$, showing that the surface on which the dome rests receives a vertical load, and that no tension ring is required under the bottom course. The reason for this sudden change is that the curvature of the profile changes at this point from $\frac{1}{R}$ to 0. It is the curvature of the profile which causes the tension—a sort of toggle action; and where this curvature changes to zero, the tension does likewise. In practice, of course, the thickness of the stone is such that the first few courses act as a conical surface, and no such sudden change occurs.

It is sometimes noted in discussions of this subject that the horizontal tension at any point is equal to the meridional stress and is of opposite sign. This is true only for a sphere, and for a point where the meridional stress and the horizontal stress act in planes at right angles to each other. For although the parallels and the meridians on a sphere always cross at right angles, if any finite element is acted on by the forces along these lines, the resultant of the two meridional thrusts will be nearly radial to the profile, and that of the two horizontal tensions will be horizontal, the third balancing force being the weight of the element. All three are in a vertical plane. The triangle of forces is a right triangle and the horizontal and radial sides never can be equal except at the equator; for as the element approaches zero, the resultant of the meridional stresses approaches a radial position, and only at the equator is this radius in the same line with the horizontal resultant of the ring stresses. Accordingly, when $\tan\theta = 0$, $\sec\theta = 1$, and $R = r$:

From Equation (4):

$$B = \frac{W}{2\,\pi\,r} - 0$$

From Equation (5):

$$A = -\frac{W}{2\,\pi\,r}$$

The formulas also account for the peculiar case of a dome of low rise, in which there is no plane of rupture, since all the stresses are compression; yet it requires a tension ring at the support. Equation (4) gives the ring com-

pression in the voussoirs just above the support, and Equation (6) gives the tension in the ring at the top of the support. The sudden change from compression to tension results from the fact that the slope changes suddenly at the support from an inclined to a vertical position, leaving the horizontal component of the thrust, otherwise unbalanced, to be taken by the structure below.

Another point not always recognized in analyses of domes is that, assuming that the line of stress is to be kept in the center of the shell, so that no vertical bending exists, the stresses at any level have to be provided for at that level, and are not affected by the shape of the superstructure, nor by tension rings or abutments in the structure below. Therefore, all the zones of the dome below the plane of rupture must have their own tension rings or abutments.

This is illustrated in the case of Dome No. 1 (Fig. 6), which has tension in the lower part of the shell. The plane of rupture—an awkward name—represents the level below which the dome has ring tension. Failure will occur at any point below the plane of rupture unless there is a tension band provided at that particular point. The spacing of the tension bands, therefore, should be close enough to enable the masonry to span from one band to the next.

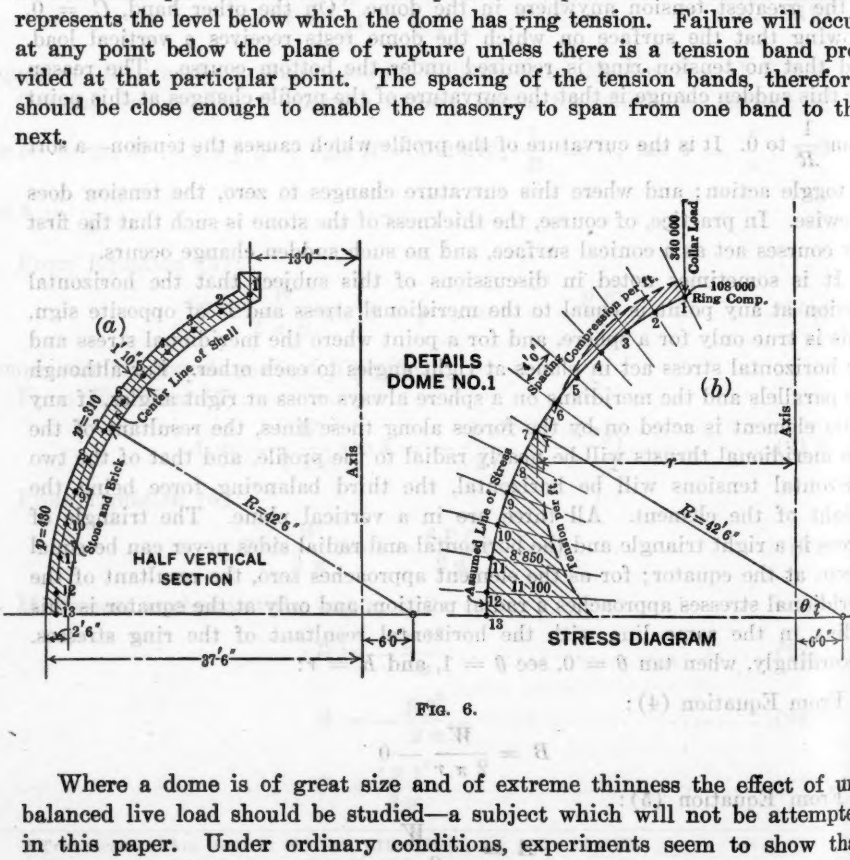



FIG. 6.

Where a dome is of great size and of extreme thinness the effect of unbalanced live load should be studied—a subject which will not be attempted in this paper. Under ordinary conditions, experiments seem to show that whatever the stresses may be under partial load, they do not greatly exceed at any point those due to full load. It seems reasonable to state, therefore, that for all ordinary cases the formulas here given are sufficient for purposes of design.

### Example—High Dome

To illustrate the application of the foregoing equations, two examples will be given from recent practice. Both are conoidal forms, one a high dome with a lantern, having tension in the lower part of the shell, and the other a flat dome with pure compression in the shell.

Dome No. 1 (Fig. 6 (*a*)) is a typical State Capitol dome with a heavy collar of masonry at the top, and inside this collar a lantern, separately carried, after the fashion of St. Paul's, London, England, by a conical interior dome. The stresses at various points are given in Table 1 for the exterior dome only; they were obtained from Equations (4) and (5).

TABLE 1.—Stresses in Conoidal Dome No. 1.

| Point. | $r$, in feet. | $p$, in pounds. | $W$, in 1000 lb. | $\tan\theta$ | $\sec\theta$ | $\frac{W}{2\pi R}(1+\tan^2\theta) - p\,r\tan\theta = B$ in pounds per foot of profile. | $\frac{W}{2\pi r}\sec\theta = A$, in pounds per horizontal foot. |
|---|---|---|---|---|---|---|---|
| 1 | 13. | 310 | 340 | 2.0 | 2.23 | 6 400 − 8 100 = − 1 700 | 9 300 |
| 2 | 16.7 | 310 | 456 | 1.58 | 1.87 | 6 000 − 8 200 = − 2 200 | 8 200 |
| 3 | 19.9 | 310 | 599 | 1.30 | 1.64 | 6 000 − 8 000 = − 2 000 | 7 800 |
| 4 | 23.0 | 310 | 766 | 1.07 | 1.47 | 6 200 − 7 700 = − 1 500 | 7 800 |
| 5 | 25.8 | 310 | 956 | 0.90 | 1.34 | 6 400 − 7 200 = − 800 | 7 900 |
| 6 | 28.3 | 310 | 1 169 | 0.75 | 1.24 | 6 750 − 6 600 = + 150 | 8 200 |
| 7 | 30.6 | 310 | 1 399 | 0.58 | 1.16 | 7 050 − 5 500 = + 1 550 | 8 400 |
| 8 | 32.5 | 310 | 1 646 | 0.47 | 1.10 | 7 480 − 4 750 = + 2 730 | 8 900 |
| 9 | 34.0 | 430 | 2 006 | 0.36 | 1.06 | 8 450 − 3 800 = + 4 650 | 10 000 |
| 10 | 35.1 | 430 | 2 381 | 0.26 | 1.03 | 9 500 − 2 850 = + 6 650 | 11 100 |
| 11 | 35.8 | 430 | 2 764 | 0.16 | 1.02 | 10 650 − 1 800 = + 8 850 | 12 500 |
| 12 | 36.4 | 430 | 3 155 | 0.07 | 1.005 | 11 900 − 800 = + 11 100 | 13 900 |
| 13 | 36.5 | 430 | 3 430 | 0.00 | 1.00 | 12 900 − 0 = + 12 900 | 15 000 |

In Fig. 6 (*b*), the values of $B$ shown in Table 1 have been plotted, indicating the various stresses. At the top is a ring stress due to the collar load of 340 000 lb.; $= \frac{340}{2\pi}\tan\theta = 108\,000$ lb. compression (Equation (6)).

At the springing, the ring stress, by Equation (6), $= 0$, since $\tan\theta = 0$. This means simply that this dome acts as a vertical load on a cylindrical support. Its internal stress is a different matter; at the springing, $B =$ 12 900 lb., which is clearly consistent with its values just above this point.

If the dome is cut by a vertical plane through the axis, the horizontal forces acting across the section must total zero. These forces are: The integrals of the tension and compression curve shown in Fig. 6 (*b*), about $+142$ and $-34$, respectively; and the ring compression at the top, $-108$. The total is zero.

To carry these stresses, it is only necessary: (*a*) to ensure that the masonry ring at the top will carry 108 000 lb.; and (*b*) to provide tension bands capable of carrying 142 000 lb., distributed throughout the lower part of the dome in proportion to the stresses shown in Table 1. These bands must be designed for permissible stretch as well as for unit stress, and with a factor to allow for live loads, such as snow, wind, and earthquakes. These matters go beyond the scope of this paper, which is the mathematical part of the design.

It is possible to dispense with tension bands by using buttresses on the outside of the dome, or by thickening the lower part sufficiently to produce a like effect. Obviously, if the tension part of the dome was replaced with a cone tangent at Point 6, this cone would be subject only to compression. By trial, a profile could be found between this cone and the given dome, along which the ring stress would be approximately zero. This fact is of interest in monumental work in which there is an objection to the use of metal ties.

*Practical Design.*—The thickness of a dome of this class may be roughly fixed by several considerations. At the top, the masonry should have a minimum thickness of, say, 16 in. for stone-faced brick, 12 in. for brick, 8 in. for Gustavino, or 6 in. for reinforced concrete. The allowable stress in compression is usually assumed to be low, not more than one-half the ordinary stresses (this may require more than the thickness previously given). The size of the dome may be such as to require a greater thickness to avoid the appearance of flimsiness. When all these considerations have been met, and the thickness at the top is fixed, a somewhat greater thickness at the bottom is assumed, and the stresses are calculated; if they are less than half the usual working stresses for the material, the design may be considered to be satisfactory. For a dome of this shape, since the effect of live load is small —perhaps 20% of the total stresses—it is permissible to allow for live and unbalanced loads by keeping to a low unit stress.

### EXAMPLE—LOW DOME

Dome No. 2 is a typical auditorium roof, with low rise and no lantern. It has no plane of rupture; the stresses are all compression. There will be no tension rings, therefore, except that at the springing.

The masonry is assumed to be 18 in. thick at the springing, varying uniformly to 6 in. at the crown. The area of the shell is about 19 500 sq. ft., and the total volume of masonry about 22 800 cu. ft. Four points will be considered, as shown on Fig. 7.

*Point (4).*—

Total weight: Masonry, 22 800 cu. ft., at 120 lb. ............ = 2 730 000 lb.

Finish and live load, 19 500 sq. ft., at 50 lb.. . = 970 000 "

Total ........................ $W$ = 3 700 000 lb.

The weight at Point (4), $p = 180 + 50 = 230$ lb. per sq. ft.; $R = 108$ ft.; $r = 72$ ft.; $\tan\theta = 0.975$; and $\sec\theta = 1.4$.

From Equation (4):

$$B = \frac{3\,700\,000}{2\,\pi\,108} \times 1.95 - 230 \times 72 \times 0.975 = -\,5\,500 \text{ lb. per ft. of profile.}$$

From Equation (5):

$$A = -\frac{3\,700\,000}{2\,\pi\,72} \times 1.4 = -\,11\,500 \text{ lb. per ft. of circumference.}$$

From Equation (6):

$$C = \frac{3\,700\,000}{2\,\pi} \times 0.975 = 575\,000 \text{ lb.}$$

From $B$ and $A$, the masonry being 18 in. thick, the following unit stresses result:

$$\text{Ring compression} = \frac{5\,500}{12 \times 18} = 25.5 \text{ lb. per sq. in.}$$

$$\text{Meridional compression} = \frac{11\,500}{12 \times 18} = 53.5 \text{ lb. per sq. in.}$$

From $C$, the stress in the tension ring is 575 000 lb.

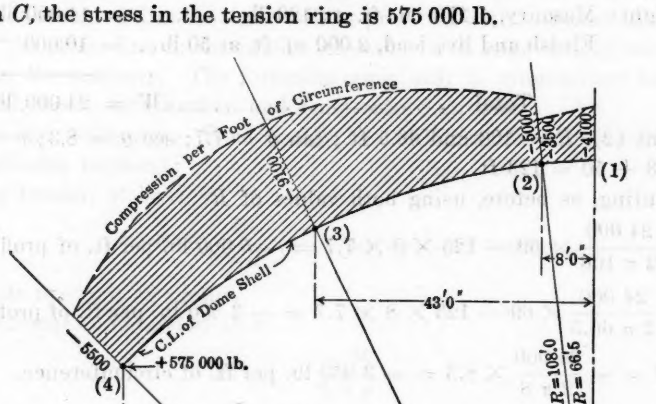



FIG. 7.

*Point (3).*—The volume of masonry above Level (3) is about 4 600 cu. ft., the area being about 5 900 sq. ft.

Weight: Masonry, 4 600 cu. ft. at 120 lb. . . . . . . . = 550 000 lb.

Finish and live load, 5 900 sq. ft. at 50 lb. = 295 000 "

Total . . . . . . . . . . . . . . . . . . . . . . $W$ = 845 000 lb.

The weight at Point (3), $p = 125 + 50 = 175$ lb. per sq. ft.; $R = 108$ ft.; $r = 43$ ft.; tan $\theta = 2.05$, and sec $\theta = 2.25$.

$$B = \frac{845\,000}{2\,\pi\,108} \times 5.05 - 175 \times 43 \times 2.05 = -9\,100 \text{ lb. per ft. of profile.}$$

$$A = \frac{845\,000}{2\,\pi\,43} \times 2.25 = -14\,000 \text{ lb. per ft. of profile.}$$

*Point (2).*—At this point, the volume of masonry above is approximately, 1 170 cu. ft., the area being about 2 000 sq. ft.

Weight: Masonry, 1 170 cu. ft. at 120 lb.......... = 14 000 lb.
Finish and live load, 2 000 sq. ft. at 50 lb.. = 10 000 "

Total ........................ $W$ = 24 000 lb.

At Point (2), $R = 106$ and 66.5 ft.; $\tan\theta = 7.7$; $\sec\theta = 8.3$; $r = 8$ ft.; and $p = 73 + 50 = 123$ ft.

Substituting, as before, using both values of $R$:

$$B = \frac{24\,000}{2\,\pi\,106} \times 69 - 123 \times 8 \times 7.7 = -5\,000 \text{ lb. per ft. of profile.}$$

$$B = \frac{24\,000}{2\,\pi\,66.5} \times 69 - 123 \times 8 \times 7.7 = -3\,500 \text{ lb. per ft. of profile.}$$

$$A = -\frac{24\,000}{2\,\pi\,8} \times 8.3 = -3\,950 \text{ lb. per ft. of circumference.}$$

The two values of $B$ represent the sudden change in the ring compression due to the change in radius (see Fig. 7). The Point (2) could not, of course, be found exactly by looking at the dome, and, similarly, no such sudden change actually occurs in the masonry. As the dome is built, the circle, of radius 108 ft., can extend some distance above Point (2) before going outside the middle third of the shell. At any rate, the formulas show that the stresses are small.

For Point (4), the stresses at the top, are given by Equations (14) and (15):

$$A = B = -p\frac{R}{2} = \frac{-123 \times 66.5}{2} = -4\,100 \text{ lb. per ft. of circumference}$$

or for a dome 6 in. thick, the stress is:

$$\frac{4\,100}{12 \times 6} = 57 \text{ lb. per sq. in.}$$

On Fig. 7, the values of the stresses are plotted radially to show in graphic form the variations of the ring compression. The integral of this curve is, of course, equal to the stress in the tension ring, 575 000 lb.

Theoretically, a dome of this type may have a minimum thickness of zero, except for the effect of live and unbalanced loads. Pending the development of a practical method of calculating the stresses due to such loads, it is reasonable to state that by using working stresses in compression of not more than one-half the usual amounts ample safety is secured. The usual thickness at the top varies from 3 in. for a 50-ft. dome to 6 in. for a 150-ft. dome, for tile, brick, or concrete, without distinction. At the springing, the thicknesses of some recent domes have been 3 in. and 5 in. for 24-ft. and 53-ft. concrete domes, respectively, and 7½ in. for a 98-ft. tile structure. A thickness should be assumed and the stresses calculated; if the dome is very flat, the stresses will be high, and the assumed thickness may have to be increased.

## Summary

In this paper, there have been developed formulas for the ring and meridional stresses in domes, and for the stress in a cylindrical support or a cylindrical lantern, at the point of junction with the dome.

The formulas for conoidal and pyramoidal domes are here repeated in full. It is assumed that the dome will be provided with sufficient reinforcement to hold the stress line to the profile selected, which may or may not lie in the center of the masonry. The formulas apply only to symmetrical loading, and disregard elastic deformation and temperature stresses.

*Formulas.—Conoids.*—For conoids the following formulas hold (positive sign indicates tension):

Ring tension at level, $x$:

$$B = \frac{W}{2\,\pi\,R}\,(1 + \tan^2\theta) - p\,r\tan\theta \qquad (4)$$

in pounds per foot of profile.

Meridional compression at $x$:

$$A = \frac{-\,W}{2\,\pi\,r}\sec\theta \qquad (5)$$

in pounds per horizontal foot.

Tension in top of a supporting cylinder, or compression in bottom of a cylindrical load, applied at the point, $x$:

$$C = \pm\,\frac{W}{2\,\pi}\tan\theta \qquad (6)$$

in pounds.

In a closed dome, pointed at the top like a Gothic arch, the ring stress at the crown approaches the value:

$$B = \frac{p\,r^2}{2\,R\sin\theta}\,(1 + \tan^2\theta) - p\,r\tan\theta \qquad (13)$$

and the meridional stress approaches the value:

$$A = \frac{-\,p\,r}{2\sin\theta}\sec\theta \qquad (12)$$

At the top of a closed dome rounded at the top:

$$A = B = -\,p\,\frac{R}{2} \qquad (14) \text{ and } (15)$$

*Formulas.—Pyramoids.*—For pyramoids the following formulas hold:

Ring tension at level, $x$:

$$B = \frac{W\,D}{n\,L\,R}\,(1 + \tan^2\theta) - p\,D\tan\theta \qquad (8)$$

in pounds per foot of rib profile.

Meridional compression at level, $x$:

$$A = \frac{-\,W}{n}\sec\theta \qquad (9)$$

in pounds per rib.

Tension in top of a supporting prism, or compression in the bottom of a prism load, applied at the point, $x$:

$$C = \pm \frac{W D}{n L} \tan \theta \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (10)$$

in pounds.

Bending in a horizontal element, 1 ft. high (fiber stress due to this is to be added to $B$):

$$\pm M = \frac{W L}{12\, n\, R} (1 + \tan^2 \theta) - p \frac{L^2}{12} \tan \theta \ldots\ldots\ldots\ldots\ldots (11)$$

in foot-pounds per foot of profile.

# THE ECONOMICS OF HYDRO-ELECTRIC DEVELOPMENT

By Daniel W. Mead,* M. Am. Soc. C. E.

To Be Presented at the Spring Meeting of the Society, Atlanta, Ga., April 9, 1924

## Synopsis

The main question involved in the consideration of any proposed hydro-electric development is: Can energy be supplied from the proposed development to a market at such time, in such quantities, and at such price, that it will command the market and assure a suitable return on the total cost of the development? The answer to this question depends on a multitude of factors any one of which may be so unfavorable as to require a negative answer to the question or, if overlooked, to result in the failure of the project.

These factors must be sub-divided, in order fully to cover the numerous questions which must be considered for a full analysis of any project. This has been attempted in a fairly complete manner in Appendices A, B, and C. Of the points discussed, the most important are, perhaps:

1.—The difficulties and expense involved to an independent development, in entering a market already served, in part at least, by power from some other source, and the resulting low price at which the hydro-electric power must often be sold.

2.—The great variations which commonly occur in the flow of streams, the difficulties in determining the average and extremes of these variations, the resulting irregularity in the power supply from such streams, the low value of the resulting irregular power supply, the necessity for auxiliary power to render the total power developed firm power, the economical point of development of such combined power, and the advantages of hydro-electric developments by power companies having a business already established.

3.—The handicap to the economical development of an independent hydro-electric project, due to the heavy cost of financing a somewhat speculative enterprise.

4.—The sources of expense to an independent development in establishing a business, due to early lack of income which necessitates deferred dividends and the payment of fixed charges and, perhaps, of operating expenses from other than income sources; the necessity of provisions to meet development expenses; and a method of estimating the financial operation and the expense of market development.

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5.—The necessity for accurate estimates of income and expense. The recognition of hazards which may develop, and the necessity of provision for such hazards by a somewhat excessive possible balance in favor of the project, or its abandonment.

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The first question of economics involved in the consideration of any hydro-electric development is that of expediency. Is it desirable that the development be undertaken?

The success of any hydro-electric development depends on the possibility of supplying energy in such manner and at such a price, that it will command a market and assure a fair return on the cost of the development. This principle is easily expressed and easily understood, but is difficult to apply to any concrete case. A favorable answer to this question depends on many factors which may be classified in a great variety of ways, but which are herein considered under the following headings:

1.—On the acquisition of suitable legal rights.
2.—On the command of a satisfactory power market.
3.—On reasonably suitable physical conditions.
4.—On favorable hydrological conditions.
5.—On the proper design of the works involved.
6.—On economical and substantial construction.
7.—On economical financing.
8.—On rapid and economical development of business.
9.—On economical management, operation, and maintenance.
10.—On accurate estimates of power output and income and of cost of promotion, financing, construction, depreciation, development, management, operation, and maintenance.

These factors will vary in importance with each possible development. Any one of them, however, may be so unfavorable as to be fatal to the economic success of any particular project. There will never be a development in which all these factors are so favorable as to leave nothing to be desired; hence, economic success will be assured only when the quantity of power available and the market are such as to provide for the expense of overcoming any and all unsatisfactory conditions.

The attempt will be made in this paper only to discuss each factor briefly and with particular reference to those elements that have proved to be of most vital importance or have been most commonly overlooked or under-estimated:

### 1.—The Acquisition of Suitable Legal Rights

The necessity of franchise or legal rights under the Federal or State laws to build and to operate a hydro-electric development is obvious and needs no further mention. Attention must be given to the State water laws which differ widely, from the English principle of riparian ownership in the East, to the entire separation of land ownership and water rights in the West.

The acquisition of flowage rights and rights of way is always of importance. With the low-head plants of the East, constructed on rivers which flow through

populous farming districts and with which considerable pondage or storage is necessary to equalize flow or to take care of the load factors, the acquisition of legal rights and rights of way may be a troublesome and uncertain process, the expense of which is difficult to determine accurately in advance. With high-head plants, however, in mountainous country, unfit for settlement and agriculture, the storage required may be comparatively small and the legal rights simple to determine and obtain.

For a private development, no condemnation proceedings are possible in most States, and site, flowage, and rights of way may be purchased only if owners are willing to sell. In some States, even public utilities are not given condemnation rights or are given only limited rights; and when the necessary properties are in unfriendly hands, exorbitant charges for property sometimes lead to an excessive cost for the project as a whole. Lands condemned often are given high valuations, as the sympathy of Courts, and especially of jurors, is usually with the private citizen and adverse to corporations.

A further point of importance, as a probable item of expense in many developments in settled parts of the country, is the excess flowage land that may have to be acquired, or the damages, real or imaginary, that may have to be paid in connection with flowage. One of the most difficult problems to solve with exactness is the height of back-water, caused by a dam under the different conditions of stream flow, and the resulting limits of flowage. Even when the flowage can be determined with fair accuracy, it is often impossible to establish the facts to the satisfaction of Court and jury. When an unusual flood occurs soon after the construction of a development, claims for damages to land, which will involve either purchase or the payment of damage, are likely to be raised. One successfully prosecuted claim of this kind may give rise to many other claims. Frequently, the expense of purchase, damages, and litigation concerning such lands becomes a considerable item.

### 2.—The Command of a Satisfactory Power Market

The difficulties of determining market conditions and the price for power which a development can command in such market, vary widely for different developments. If the development is a part of an operating system, the market will already have been established, the price obtainable for power will be certain, and all market conditions known. However, when a new development has to enter a market already supplied, in part at least, by other forms of power, and especially where power is furnished from a central station operating profitably under the conditions that have previously existed, it often becomes exceedingly difficult to induce the operating company to buy and distribute power from the hydro-electric plant at a price which will be equitable and profitable to the new development. Almost uniformly the price that can be obtained under these conditions will be much below any estimated price for power that the hydro-electric interests are likely to anticipate.

When a market must be developed for a new hydro-electric plant, extended examinations and investigation are necessary. Is the demand for power sufficient to warrant the development? Can a price for power be obtained that will warrant the necessary investment?

A census of power users will determine the amount of power used in the territory considered. The determination of the price at which power can be sold is a more difficult problem. The actual value of power in any community where power is a necessity, is the cost of developing such power by the means ordinarily used, or which ordinarily would be used under the conditions and in the quantities used, or to be used, in the particular location. This is the upper limit at which power from a new hydro-electric development can be sold, and such price can be obtained only when the market is practically undeveloped by other sources. The lower limit is the actual cost of the power to the hydro-electric company. Between these limits, there is frequently a wide range in values. The higher price must not be so great as to discourage the use of power or as to induce large users to undertake its generation rather than to contract for its purchase.

When the market is already supplied with power generated by fuel-burning plants, the sale price for hydro-electric power must be lower than the cost of fuel-generated power, otherwise, users will continue to generate their own energy. With the developed market, the higher price previously referred to is so high that, for commercial reasons, such a rate cannot obtain. At this price, the customers would pay a rate equal to the actual cost of power generated by their own plant, and there would be no inducement for them to become customers of the hydro-electric development. The higher price is, therefore, a commercial impossibility. The lower rate, the actual cost of hydro-electric power, is equally impossible, for such rates would not encourage hydro-electric development. The public, including all consumers of power generated by a hydro-electric plant, will from commercial necessity receive a portion of the benefit from the use of such power, from the commercial conditions that follow its development. Rarely, if ever, can a hydro-electric development obtain the higher return for its output. Such a development, as a rule, must supply power to a market which is, partly at least, supplied with power from some other source. Investments in power-generating machinery of some kind have been made. Fixed charges have been incurred, and, in order to introduce its product, the hydro-electric development must sell power below the station cost of fuel-generated power, and not on the basis of fixed charges plus operating expenses of such plants, which has been the true measure of the actual value of power to the date of its introduction from the new source. Only where the market developed is entirely new, and where no fixed charges have been entailed for previous power-plant installation, can a hydro-electric development realize from the sale of power even a part of the fixed charges of the steam plant. Even under such conditions, a material reduction must be made, in order to induce customers not to install isolated plants of their own for the production of such power, but to purchase the power developed from the hydro-electric plant.

For example, consider a community of 30 000 or 40 000 in the interior of the country. A well planned steam-electric plant working under the conditions which ordinarily obtain in such communities, can develop power at its switchboard for about 1.6 cents per kw-hr. Of this amount, approximately 0.5 cent per kw-hr. is fixed charge and 1.1 cents per kw-hr. is operating cost.

Assume that hydro-electric power can be delivered at the city stations at a cost of 0.6 cent per kw-hr. If a hydro-electric company sells its output to the steam-electric company, it will be found that the fixed charges, maintenance, and some of the operating expenses on the steam plant, will continue, as the steam plant must be maintained ready for almost instant use, in case of failure of the transmission line, in order to assure the service to which the community is entitled. There is no immediate economy to the steam-electric company in the purchase of water-generated power unless it can be purchased at less than the operating cost of steam power. Consequently, to effect a sale, the water power must be sold at 0.7 to 1 cent per kw-hr., delivered at the customer's switchboard. The higher price would probably not be regarded as attractive to the steam-electric company, and the lower price would seldom be attractive to the hydro-electric company and might involve a direct loss, unless the total current sold to all customers constitutes a large proportion of the capacity of the plant. If a direct combination can be effected between the water-power company and the steam-electric company, the steam plant may be utilized as an auxiliary to the water power, and the whole value of the output utilized by the combined interests. This combination is usually the only way in which a larger net profit can be obtained from a water-power development. The best results can be obtained only by combination with an industry or market already developed in which the power can be utilized at its true market value.

In considering the influence of the market, it should also be noted that the load factor, on which power must be sold, is an important element in the cost of hydro-electric development. A combined load approximating a load factor of 100%, which is seldom possible, involves a minimum of expense in machine installation. A load factor of 50% practically doubles the cost of machinery and greatly increases the cost of the power-house, and a load factor of 33⅓ or 25% triples or quadruples the cost of the machine installation.

### 3.—Reasonably Suitable Physical (Topographical and Geological) Conditions

Physical conditions at the dam and power-house sites are sometimes easily determined, but adequate examination frequently entails considerable delay and expense. At the preliminary inception of a project, it may be exceedingly important to limit to a minimum the expenditures for examination, and proper physical examination may be delayed until other conditions seem to demand immediate and favorable action, and the project is launched without a complete and satisfactory physical examination having been made. If unfavorable conditions, not readily foreseen, develop during construction, the results may be disastrous, if not financially fatal.

The physical conditions at the site of the dam, the station, and appurtenances are among the most important factors affecting the cost of construction. A poor foundation may prove fatal to the success of the project, as in the case of the municipal plant at Austin, Tex., or may involve a large and burdensome expense as, for example, the Hales Bar development on the Tennessee River. Thorough and detailed examination prior to construction would probably have led to a knowledge of these conditions, and to a corre-

sponding change in location or in plans, or the abandonment of the project. The works must be designed to fit and to develop the physical conditions to the best advantage, and the physical conditions must be studied with regard to the particular works designed to develop them.

## 4.—Favorable Hydrological Conditions

Where hydro-electric plants are entitled to take a limited water supply from great rivers, the conditions of stream flow may be definite and well established. In the development of any ordinary stream to the point of maximum economy, the great variations of flow which normally occur become important matters. These variations in stream flow, the consequent variations in head, and the resulting possible power output, must be considered with reference to market conditions.

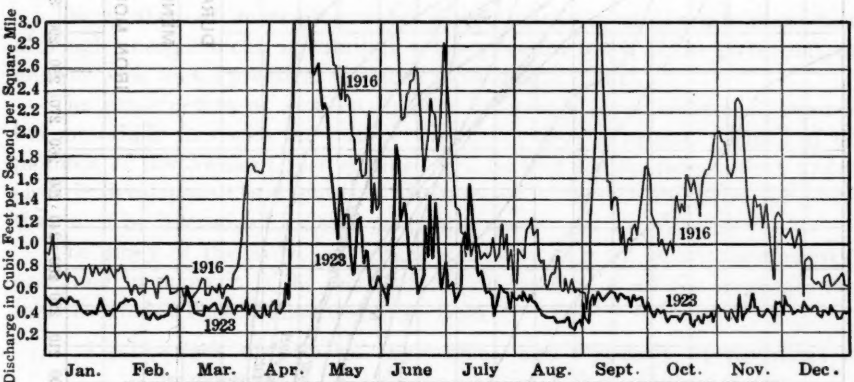
Although there are some notable exceptions, the market in general requires the continuous delivery of power. The annual load will vary somewhat due to the season. Some loads are less in the winter and greater in the summer; others are greater in winter and less in summer; and some are subject to hourly, daily, or to seasonal variations. These loads vary somewhat from year to year, usually increasing in growing communities, and fluctuating according to commercial activities; but the general form of load curve remains quite the same in character for a given community.

The water power which can be developed from a given stream is also subject to considerable variation, due to the character of the stream and the climatic conditions peculiar to each particular drainage area. Usually, there are one or two seasons of high water and of low water during each year. The high water in most streams will occur generally in the spring, and sometimes, also, in the fall. The dry season of summer, together with the demands of vegetation, produces low water, and in northern streams, the freezing of the sources commonly results in even lower water during January and February. In addition to these variations, the years differ widely. Occasionally, there are years of fairly continuous high flow whereas other years have equally continuous low water, and these periods rarely bear any relation to the demands for power.

No one unfamiliar with hydrological studies will realize the great variations in the flow of streams from year to year. Fig. 1 shows the daily stream flow of a single river (not an extreme case) for the maximum and minimum years during the last eleven years.

All hydrological studies with reference to power output can be made to greater advantage by platting the data in the form of duration curves, which will show the duration of each intensity of flow for each year without reference to date of occurrence (Fig. 2). From all the data available, for the period of records, an average duration curve can be platted (Fig. 2) from which the average quantities of power that could have been developed from the stream during the period of record can be determined. From a study of Fig. 2, using the flow in cubic feet per second per square mile of drainage area as the basis for power study, it will be noted that during the year of minimum low-water flow (1919), power would have been available on the stream

in question for 98.5% of the time, to a flow corresponding to 0.25 sec-ft. per sq. mile; during the lowest year (1923) to 0.3 sec-ft.; during the average year to 0.35 sec-ft.; and during the highest year to 0.51 sec-ft. In general, therefore, the hydro-electric development is confronted with the conditions of a superabundance of power at certain seasons and for certain years, and with a deficiency of power at other seasons and for other years. These conditions must be met in one of the following ways:



DAILY HYDROGRAPH OF YEARS OF MAXIMUM AND MINIMUM FLOW

FIG. 1.

*First.*—By the development of the stream only to a minimum flow. In general, this will entail so great an expense for such a small amount of power output as to render any project based on such a flow entirely uneconomical.

*Second.*—By the sale of firm or primary power to certain customers, and of surplus or secondary power to other customers who can use power when it is available, or who can utilize their own power plants or other sources of power during periods when power from the hydro-electric plant is deficient.

*Third.*—By the establishment of fuel-burning auxiliary stations from which power will be available whenever a water shortage occurs. By this method, the power from a hydro-electric development can be made firm power under all conditions of river flow for which the development is designed.

*Fourth.*—By the construction of reservoirs to store the flood-waters for use during periods of low flow, thus often substantially increasing the minimum power output.

*Fifth.*—By interconnecting hydro-electric plants on streams with low water occurring at different seasons of the year.

*Sixth.*—By a combination of the methods outlined under the third, fourth, and fifth items.

Surplus power is often of comparatively small value and, if it can be sold at all, can command only a low price, whereas firm power is uniformly of much greater value. For example, for many years the grinding of pulpwood has been considered one of the ways of using surplus power to advantage. During the season of high water, an excess of pulp is manufactured, lapped, and stored for use during low water. This process requires the product to be pressed

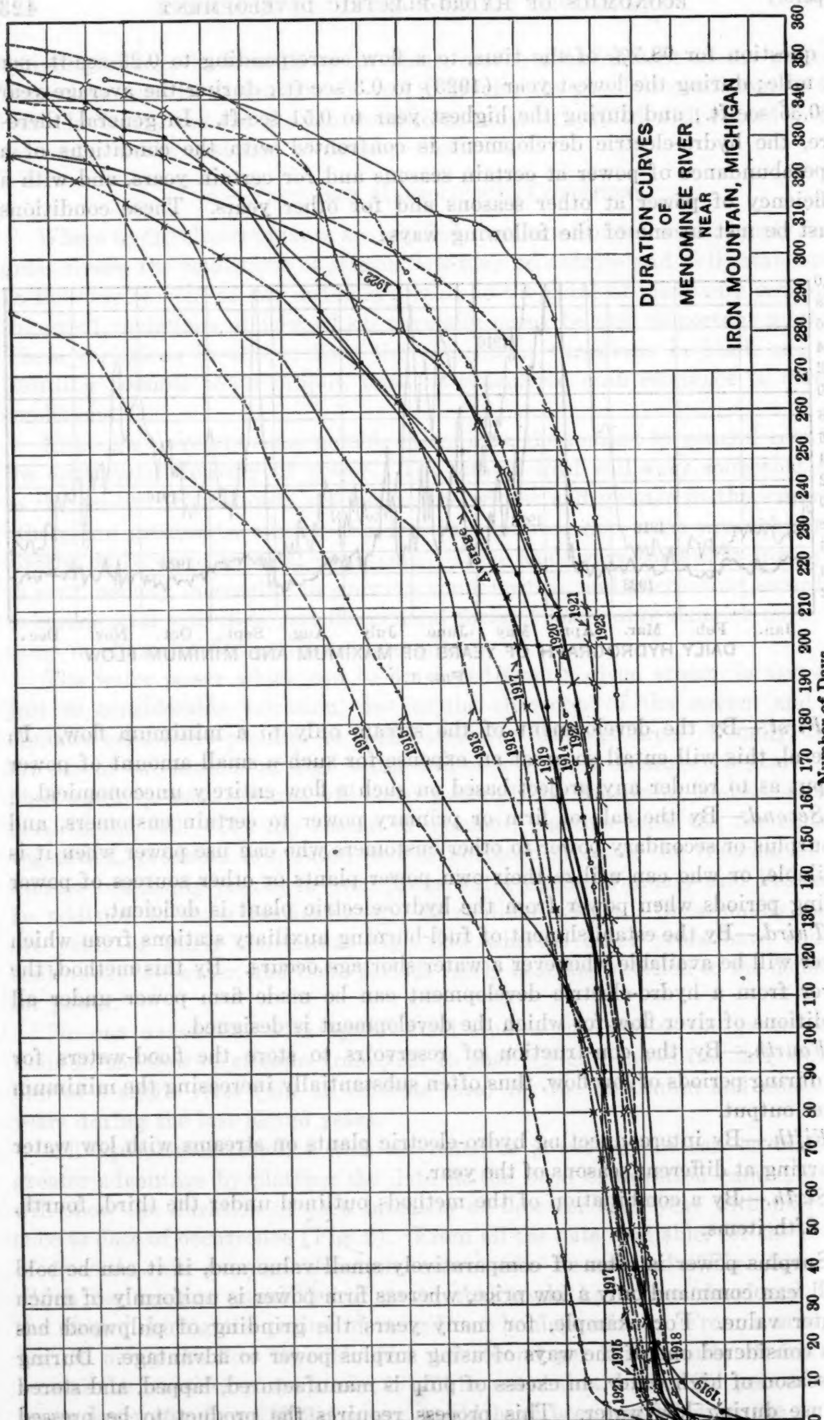

DURATION CURVES
OF
MENOMINEE RIVER
NEAR
IRON MOUNTAIN, MICHIGAN
Number of Days
Discharge in Cubic Feet per Second per Square Mile
Average
1913
1914
1915
1916
1917
1918
1919
1920
1921
1922
1923

FIG. 2.

dry, stored, and beaten, when used for paper-making, involving an expense for lapping, handling, storing, deterioration in storage, re-handling, and beating, which amounts to $5 to $7 per ton. Therefore, even if surplus power for grinding can be obtained as low as 0.35 cent per kw-hr., the paper-mill can often better afford to pay from 0.75 to 0.95 cent per kw-hr. for firm power for grinding the pulp as it is needed, pumping it to the paper machines without additional processes.

The method of developing water power in excess of the minimum flow, although uneconomical, as entailing a low price for much of the power output, has yet been found practical for some independent plants.

The construction of auxiliary plants involves a large additional expense, but materially increases the value of the hydro-electric output. Which of the methods of development just outlined will be best for an independent hydro-electric development is a question of character of market and price for power that must be determined for each particular case.

The study of the hydrology of the stream is of vital importance in every case, and in many it is the most difficult to analyze of all the economic factors involved. These difficulties arise from:

1.—The lack of data on the stream, and especially at the point of development.

2.—The inaccuracy of data.

3.—Lack of knowledge as to whether data available even when accurate, fairly represent the average future stream flow.

Although a large amount of stream-flow data are being obtained by the U. S. Geological Survey, it often happens that data are not available for the particular stream studied and they are still less frequently available at the particular site of a proposed development. Even if information for a period of years is available, the accuracy of stream-flow data remains to be determined; and if the accuracy is unquestioned, there still remains the problem of how closely the record for a given term of years represents the flow to be expected during the life of the development.

When flow records for the site are not available, the data for other sites must be used as a basis for an estimate at the site under consideration. If no data are available for the stream, data from other neighboring streams must be used. The transfer of data or the determination of the flow of one stream from that of another is of doubtful accuracy, even if the streams are closely adjoining. This great variation in adjacent streams is illustrated by Fig. 3, which shows the comparative daily hydrographs of the Hudson, Oswego, and the Genesee Rivers in the State of New York.

Table 1 shows the possible inaccuracies in the data, derived from even a considerable period of records, when averaged for forecasting the future. This table shows the maximum and minimum variations in average annual stream flow from the means of certain fairly extended records, and the more pertinent data, for the study of water power, of the variations from average flow for the minimum six months of the flow of various streams. It will be noted from Table 1 that the lower part of the average duration curve

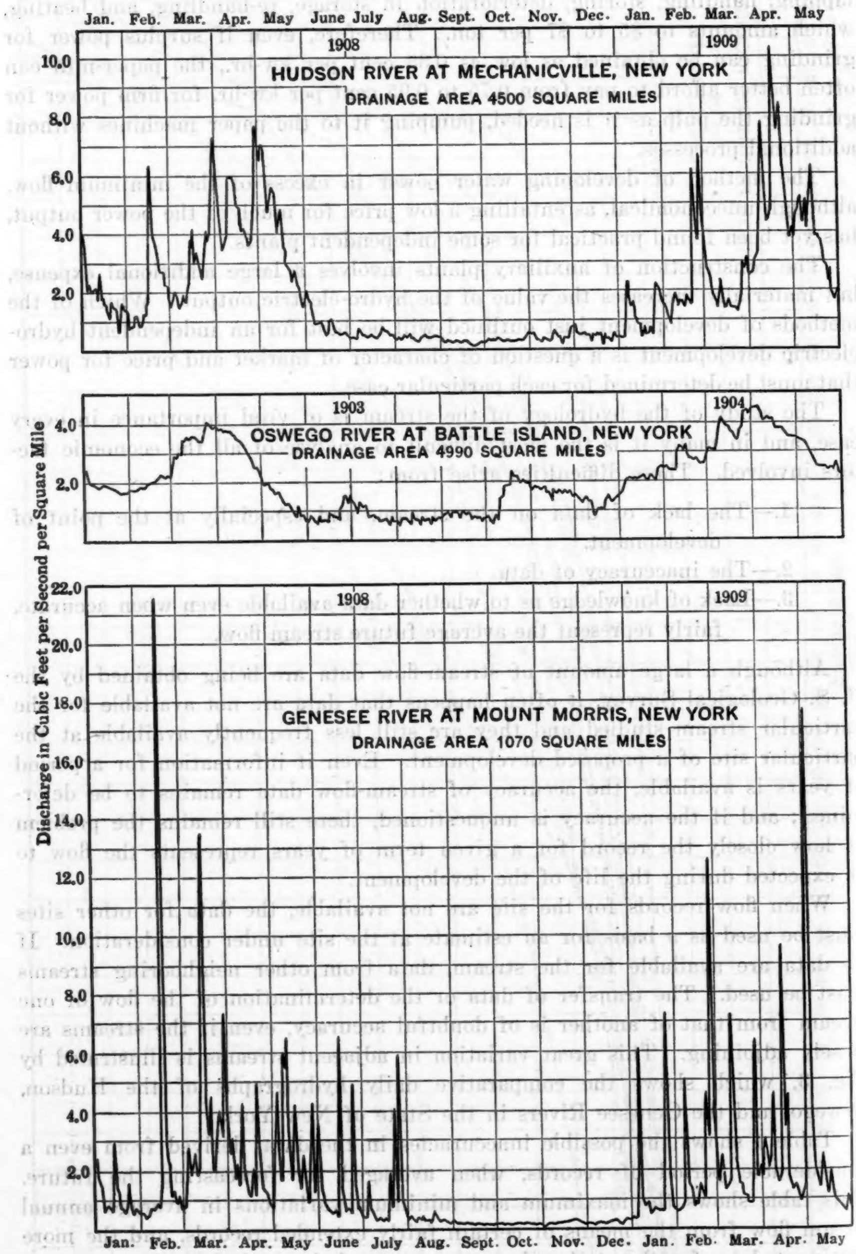

Jan. Feb. Mar. Apr. May June July Aug. Sept. Oct. Nov. Dec. Jan. Feb. Mar. Apr. May
1908
1909
HUDSON RIVER AT MECHANICVILLE, NEW YORK
DRAINAGE AREA 4500 SQUARE MILES
10.0
8.0
6.0
4.0
2.0
1903
1904
OSWEGO RIVER AT BATTLE ISLAND, NEW YORK
DRAINAGE AREA 4990 SQUARE MILES
4.0
2.0
Discharge in Cubic Feet per Second per Square Mile
1908
1909
GENESEE RIVER AT MOUNT MORRIS, NEW YORK
DRAINAGE AREA 1070 SQUARE MILES
22.0
20.0
18.0
16.0
14.0
12.0
10.0
8.0
6.0
4.0
2.0
Jan. Feb. Mar. Apr. May June July Aug. Sept. Oct. Nov. Dec. Jan. Feb. Mar. Apr. May

Fig. 3.

TABLE 1.—MAXIMUM AND MINIMUM VARIATION IN AVERAGE STREAM FLOW FROM MEAN RECORD.

| Rivers. | Years of record. | Number of years. | Depth, in inches, mean flow. | 1-Year Departure. | | 5-Years Departure. | | 10-Years Departure. | | 15-Years Departure. | | 20-Years Departure. | | 25-Years Departure. | | 30-Years Departure. | | 35-Years Departure. | | 40-Years Departure. | | 45-Years Departure. | | 50-Years Departure. | | 55-Years Departure. | | 60-Years Departure. | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | + | − | + | − | + | − | + | − | + | − | + | − | + | − | + | − | + | − | + | − | + | − | + | − | + | − |
| Lake Champlain Outlet, Richelieu River | 76 16 | 41 | 20.46 | 29.10 | 29.48 | 17.88 | 15.46 | 8.34 | 9.00 | 7.40 | 1.14 | 6.25 | 1.48 | 5.35 | 0.84 | 3.65 | +0.98 | 2.04 | +0.29 | 0.62 | 0.51 | .... | .... | .... | .... | .... | .... | .... | .... |
| Presumpscot River, at Outlet at Sebago Lake, Maine | 87 19 | 33 | 20.55 | 46.92 | 52.74 | 27.62 | 25.34 | 11.35 | 20.13 | 11.45 | 14.42 | 6.60 | 9.08 | 0.70 | 7.32 | 0.70 | 2.40 | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... |
| Sudbury River, at Framingham, Mass. | 75 16 | 42 | 20.92 | 70.89 | 48.83 | 37.32 | 36.04 | 18.26 | 24.52 | 16.78 | 15.44 | 14.96 | 6.98 | 9.22 | 6.64 | 8.60 | +0.68 | 4.40 | 0.76 | 0.39 | 0.34 | .... | .... | .... | .... | .... | .... | .... | .... |
| Lake Cochituate Basin, near Cochituate, Mass. | 64 18 | 55 | 19.16 | 89.25 | 53.08 | 46.00 | 31.05 | 18.15 | 20.04 | 13.50 | 16.23 | 11.80 | 8.50 | 9.80 | 7.93 | 6.60 | 2.08 | 6.05 | 4.12 | 6.05 | 3.80 | 3.95 | 2.92 | 0.83 | 1.20 | .... | .... | .... | .... |
| Merrimack River, at Lowell, Mass. | 49 60<br>67 16 | 62 | 26.74 | 29.24 | 29.40 | 21.00 | 19.41 | 16.45 | 13.80 | 11.51 | 10.44 | 8.45 | 8.08 | 6.85 | 7.22 | 5.80 | 6.14 | 2.42 | 5.35 | 0.90 | 4.52 | 0.71 | 2.28 | 0.48 | 0.82 | 0.00 | 0.86 | 0.00 | 0.22 |
| Merrimack River, at Lawrence, Mass. | 50 08 | 59 | 22.62 | 30.78 | 37.27 | 21.82 | 18.34 | 10.32 | 15.34 | 5.70 | 9.31 | 5.62 | 6.76 | 5.12 | 5.08 | 3.52 | 3.93 | 1.71 | 4.59 | 2.30 | 2.34 | 1.07 | 2.34 | 0.75 | 1.85 | 1.10 | 1.18 | .... | .... |
| Merrimack River, at Lawrence, Mass. | 80 18 | 39 | 20.02 | 44.70 | 46.79 | 33.50 | 24.10 | 15.05 | 19.80 | 11.56 | 12.16 | 11.40 | 5.02 | 7.97 | 4.17 | 3.67 | 0.28 | 1.32 | 0.48 | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... |
| Croton River, at Old Croton Dam, N. Y. | 68 21 | 54 | 22.57 | 51.70 | 48.20 | 27.95 | 26.36 | 16.25 | 17.68 | 9.91 | 14.14 | 10.00 | 9.97 | 8.55 | 7.27 | 6.60 | 5.94 | 5.98 | 1.51 | 3.05 | 1.68 | 0.83 | 1.99 | 0.00 | 1.77 | .... | .... | .... | .... |
| Tohickon Creek, at Point Pleasant, Pa. | 84 13 | 30 | 27.48 | 54.10 | 45.15 | 21.80 | 16.84 | 13.20 | 12.65 | 4.90 | 10.36 | 2.50 | 8.20 | 1.00 | 3.22 | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... |
| Lake Ontario | 60 07 | 48 | 12.01 | 19.25 | 24.73 | 14.65 | 15.48 | 9.75 | 12.08 | 7.25 | 9.82 | 5.60 | 7.34 | 4.82 | 4.24 | 4.00 | 3.58 | 3.75 | 3.16 | 1.40 | 2.33 | 0.32 | 1.08 | .... | .... | .... | .... | .... | .... |
| Lake Superior | 60 07 | 48 | 14.62 | 23.65 | 20.66 | 14.10 | 14.84 | 10.05 | 9.80 | 8.28 | 8.00 | 7.50 | 6.84 | 5.68 | 6.30 | 4.15 | 5.68 | 1.44 | 5.68 | 0.95 | 1.78 | 0.18 | 1.16 | .... | .... | .... | .... | .... | .... |
| Wisconsin River, at Merrill, Wis | 04 20 | 17 | 16.78 | 50.05 | 42.97 | 19.60 | 13.52 | 8.15 | 7.15 | 1.90 | 6.44 | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... |
| Tennessee River, at Chattanooga, Tenn. | 75 17 | 43 | 24.82 | 49.75 | 49.48 | 19.45 | 18.49 | 14.21 | 7.57 | 8.73 | 6.97 | 5.96 | 6.61 | 3.70 | 6.24 | 2.65 | 3.64 | 2.88 | 1.53 | 0.00 | 1.73 | .... | .... | .... | .... | .... | .... | .... | .... |
| Tennessee River, at Florence, Ala. | 95 17 | 23 | 22.69 | 31.50 | 44.64 | 14.00 | 11.18 | 3.10 | 2.67 | 3.29 | 1.66 | 1.00 | 2.05 | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... |
| Coosa River, at Riverside, Ala. | 97 16 | 20 | 22.65 | 38.65 | 57.93 | 18.00 | 11.78 | 8.36 | 2.50 | 4.51 | 1.53 | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... |
| Columbia River, at The Dalles, Ore. | 79 16 | 38 | 13.03 | 47.83 | 42.77 | 27.98 | 20.06 | 16.08 | 13.91 | 7.02 | 11.70 | 7.49 | 8.24 | 6.49 | 4.63 | 3.42 | 4.71 | 1.35 | 3.10 | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... |
| San Gabriel River, at Yuma, Ariz. | 96 18 | 23 | 10.60 | 178.70 | 92.17 | 62.45 | 74.05 | 47.45 | 53.96 | 33.20 | 17.36 | 9.80 | 5.20 | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... |
| Red River of the North | 82 19 | 38 | 1.09 | 109.90 | 74.22 | 35 35 | 50.28 | 22.45 | 25.42 | 17.85 | 9.76 | 11.40 | 4.23 | 9.60 | 3.30 | 5.00 | 7.00 | 3.12 | 7.92 | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... |
| Colorado River, at Yuma, Ariz. | 79 17 | 39 | 14541* | 146.20 | 81.23 | 98.00 | 64.88 | 76.90 | 56.66 | 69.00 | 51.35 | 45.14 | 47.14 | 28.90 | 41.20 | 16.46 | 23.04 | 7.78 | 10.12 | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... |
| Colorado River, at Austin, Tex. | 99 22 | 24 | 0.96 | 187.50 | 77.06 | 56.25 | 52.06 | 27.10 | 31.24 | 6.25 | 21.87 | 0.00 | 17.70 | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... |

*Cubic feet per second.

## TABLE 1.—(*Continued*).

VARIATIONS OF FLOW FOR THE MINIMUM SIX MONTHS.

| Rivers. | Years of record. | Number of years. | Depth, in inches, mean flow. | 1-Year Departure. + | 1-Year Departure. − | 5-Years Departure. + | 5-Years Departure. − | 10-Years Departure. + | 10-Years Departure. − | 15-Years Departure. + | 15-Years Departure. − | 20-Years Departure. + | 20-Years Departure. − | 25-Years Departure. + | 25-Years Departure. − | 30-Years Departure. + | 30-Years Departure. − | 35-Years Departure. + | 35-Years Departure. − | 40-Years Departure. + | 40-Years Departure. − | 45-Years Departure. + | 45-Years Departure. − | 50-Years Departure. + | 50-Years Departure. − | 55-Years Departure. + | 55-Years Departure. − | 60-Years Departure. + | 60-Years Departure. − |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Merrimack River, Lawrence, Mass. | 80 19 | 39 | 4.88 | 72.55 | 50.41 | 50.60 | 32.78 | 21.70 | 18.03 | 15.78 | 12.70 | 12.70 | 8.00 | 9.00 | 6.56 | 4.30 | 0.40 | 1.64 | 0.40 | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... |
| Croton River, at Old Croton Dam, N. Y. | 68 21 | 54 | 8.94 | 110 16 | 76.25 | 49.08 | 42.03 | 36.75 | 33.42 | 23.55 | 27.70 | 27.22 | 16.29 | 16.71 | 9.91 | 13.48 | 5.10 | 10.56 | +0.14 | 5.51 | 1.34 | 1.17 | 2.52 | 0.05 | 2.64 | .... | .... | .... | .... |
| Tennessee River, at Chattanooga, Tenn. | 75 17 | 43 | 6.43 | 79.69 | 52.91 | 30.71 | 24.46 | 14.55 | 12.64 | 6.81 | 8.24 | 4.12 | 4.56 | 6.78 | 4.10 | 3.82 | 3.32 | 2.72 | 0.22 | 0.00 | 1.46 | .... | .... | .... | .... | .... | .... | .... | .... |
| Columbia River, at The Dalles, Ore. | 79 16 | 38 | 2.91 | 43.40 | 27.78 | 14.17 | 14.02 | 9.35 | 8.53 | 4.19 | 7.15 | 4 19 | 2.34 | 4.19 | 2.34 | 1.79 | 3.02 | 0.41 | 3.10 | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... |
| Colorado River, at Austin, Tex. | 99 22 | 24 | 0.33 | 438.25 | 78.46 | 84.60 | 60.00 | 38.45 | 47.69 | 10.77 | 35.38 | 4.60 | 26.15 | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... |

shown in Fig. 2 might vary approximately ± 18% on the basis of the Merrimack River, ± 35% on the basis of the Croton River, ± 13% on the basis of the Tennessee River, ± 8% on the basis of the Columbia River, and ± 40% on the basis of the Colorado River of Texas, according to the location and character of the stream.

Comparative studies of rainfall for the years during and previous to the years of stream-flow record will often indicate with considerable accuracy the possibility of great variation from the recorded average flow of any stream.

Owing to lack of space, discussion of hydrology must be confined to the foregoing statements, which are designed merely to call attention to the economic importance of the subject and to the difficulties frequently involved in reaching correct conclusions from the data usually available.

After a duration curve is established, that is believed fairly to represent the future average flow of the stream, the economical point of development can be determined approximately by a method devised by F. E. Turneaure, M. Am. Soc. C. E. This analysis is shown in Appendix D.

### 5.—The Proper Design of the Works Involved

As this factor cannot adequately be discussed in a paper of reasonable length, only a few important points will be mentioned; but other points are noted in the more complete outline of Appendix A. The factors outlined in the beginning of this paper of necessity overlap to a certain extent; under the heading, "Reasonably Suitable Physical Conditions", the question of foundations has already been briefly considered. This question is one of judicious selection as well as physical condition, as conditions sometimes vary greatly within comparatively narrow geographical limits.

It is of great economic importance that structures shall be designed to provide for the maximum floods likely to occur, and this again reverts to the study of the hydrology of the stream. Occasional floods may occur that greatly exceed those recorded, even in long-time records of maximum stream flow. To illustrate this point, reference may be made to Fig. 4, which shows a view of the reservoir and dam of the Oklahoma City Water-Works. A flood of about 133 000 cu. ft. per sec. washed around the end of this dam during October, 1923. The dam and flood-control works, which have a drainage area of 12 000 sq. miles tributary to them, were designed to pass about 18 000 cu. ft. per sec. This was much in excess of any recorded flow, the records being very limited. A study of the hydrology of the stream leads to the conclusion that no flood equal to that of October, 1923, has occurred in fifty years, but that even a greater flood must be expected.

In the provision for floods, a point of much economic importance should be noted. If excessive floods are provided for by an overflow spillway, flowage lands to a considerable height above the crest of the dam must be purchased to be utilized only during floods. With many plants, these lands may be utilized to increase the operating head and pondage by the use of gates in the dam instead of an open spillway. These gates will keep the head-waters at a maximum elevation during low water and provide high discharge capacity at times of flood.

The provision of pondage so that during days of average flow, water may be stored during periods of low power consumption and used during the peak demand without too great a reduction in head, is another important economic problem, especially with a low load factor. In general, it may be stated that plants should be designed with the human equation always in mind. As far as possible, the arrangement should be such that the right thing is the easiest thing to do. Considerable expense is warranted if operating costs can be reduced. With this principle in mind, automatic, semi-automatic, and remote controlled stations must be mentioned.

Efficiency in operation is also of much importance. High efficiency is frequently sacrificed in order to obtain large power. Installations must be designed to obtain maximum efficiency over the normal range of operation rather than at the maximum load. Often 2 or 3% in over-all efficiencies amount to more in the aggregate than interest and depreciation on the entire cost of machinery. There is a tendency at present to sacrifice turbine efficiency for high-speed generators in order to keep down the initial cost of machines. This may not be warranted when capital costs, operating conditions, and the amount and cost of output are considered.

It is evident that the previous discussion touches only a few of the points which must be considered in the economical design of hydro-electric developments, and that study of all the elements of design is essential for the best economy.

### 6.—Economical and Substantial Construction

This subject, although worthy of extended discussion, will be dismissed with only a single paragraph. The cost of construction can usually be reduced by a complete knowledge of the actual conditions. Any doubt concerning the contingencies of construction will add to the contractor's estimate of cost and, if the work is done by day labor, will make the question of construction equipment doubtful, and is likely to entail expensive delays. In the construction of hydro-electric plants, adjustments must continually be made between reasonable security and unnecessary expense, and few criteria are available outside extensive and broad experience. Safety and economy are both essential and neither should be sacrificed.

### 7.—Economical Financing

Uneconomical financing has been one of the greatest handicaps in the economical development of independent hydro-electric plants. Many projects have been financed at low rates of interest only by such heavy discounts on the securities that the interest charges have absorbed most of the income in excess of other fixed charges and operating costs. In this way, all possible profits, at least for a long term of years, have been absorbed in the financing.

It is comparatively easy for an operating industry that is showing fair returns on the investment, to obtain the money necessary for reasonable expansion, or for its current business. The cost of financing a corporation the property and business of which are both a question of future development, and necessarily somewhat speculative, is more expensive. A hydro-electric com-

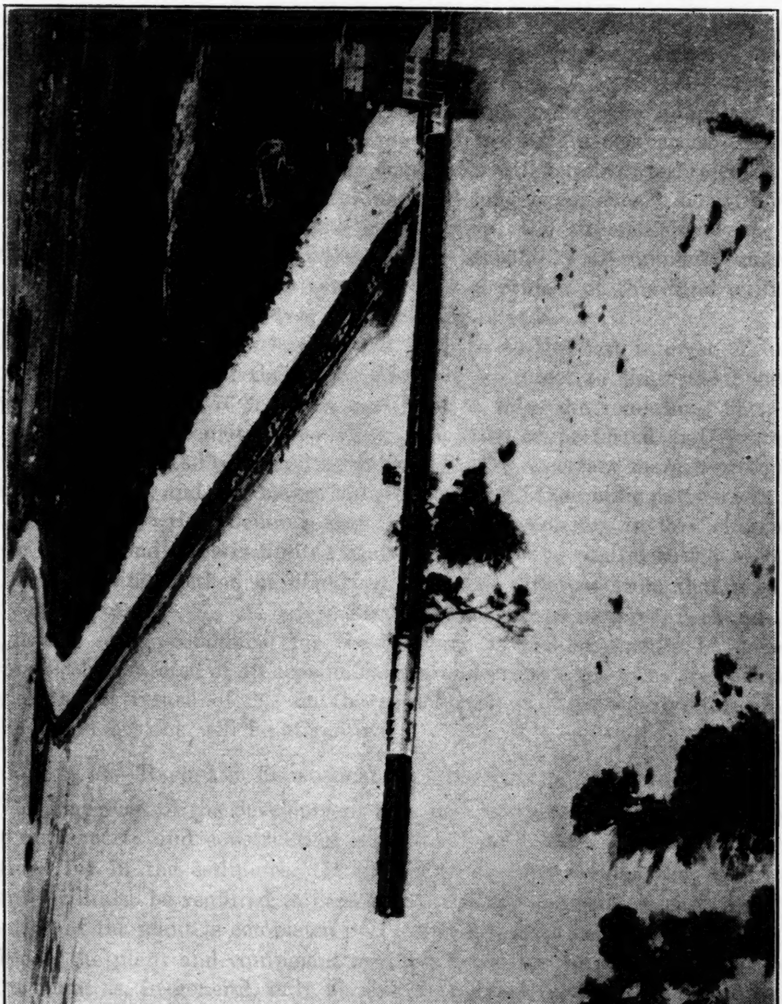
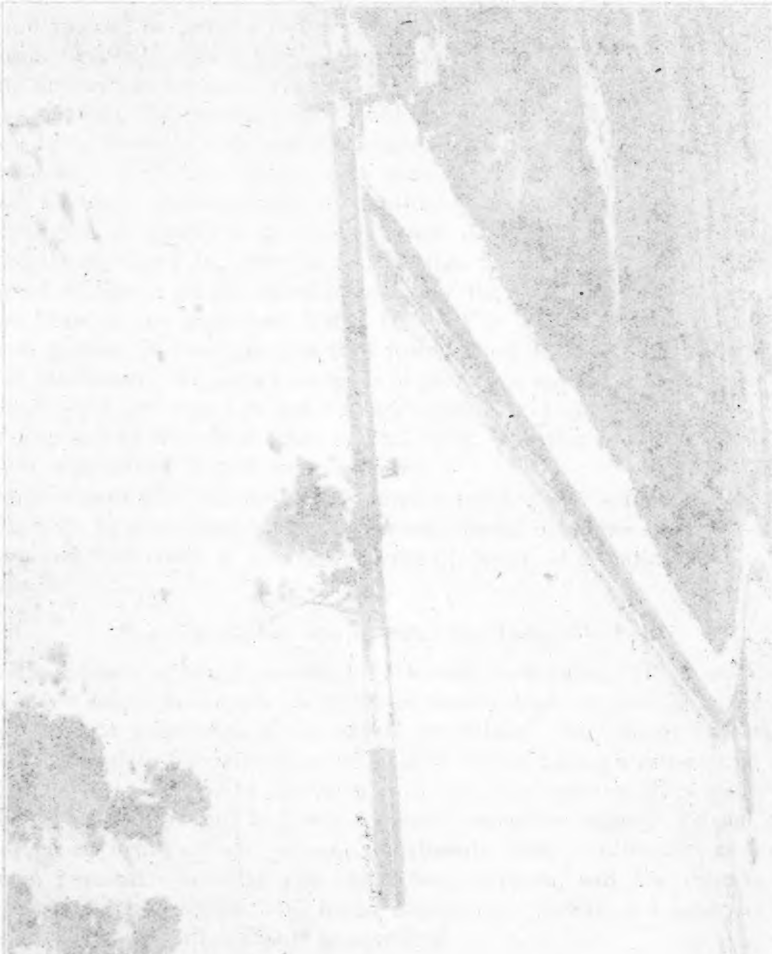


FIG. 4.—VIEW OF RESERVOIR AND DAM OF CITY WATER-WORKS, OKLAHOMA, OKLA.



pany can seldom obtain from its own stockholders the necessary funds to complete its project unless a market at a known price is assured, and then, only if the project is comparatively small and high returns are believed to be certain.

If the cost of development is several million dollars, the financing of a project is possible only through the agency of investment companies. Before purchasing such securities, reputable companies will require the project to be examined by men of the highest ability and long experience, who will pass on the technical features of the construction, the expense involved, the market available, the legal features, and the probability of complete commercial success. Such companies will not assist a project of this kind without fair returns both for themselves and for their clients.

Many regulatory bodies have found it to be to the best interest of both the investing public and the stockholders of a project to limit the bonded indebtedness to about 75% of the cost, and to raise the remaining 25% by the sale of junior securities, usually in the form of preferred and common stock. The sale of the preferred stock provides the necessary margin of equity back of the bonds, and also makes the employees and the public partners in the enterprise. The writer believes that the greatest economy in financing will obtain if the bond issue is limited to about 50% of the capital cost.

The preceding method of financing, although different from that used in many of the earlier projects before State regulation was in force, is sound and will be the most economical for the industry in the long run. If, as can reasonably be expected of an economically sound project, the rates are sufficient to yield a net return of 8% on the actual cost of the property, the profits to the common stock will be attractive.

### 8.—Rapid and Economical Development of Business

The high cost of the development of a new business is rarely appreciated. Cost of property and construction is obvious and usually will be reasonably provided for in the estimates. It should be equally obvious that time and expense will also be required to create an independent business after the construction of the plant is completed and before a market can be developed.

When the plant and equipment are first ready for operation, the business of the plant is, in general, only partly, if at all, developed. It is yet to be demonstrated whether the project was advised, designed, and constructed properly. Faults must be corrected, weaknesses remedied, machinery adjusted, a staff trained, and a business established. The plant must operate, even at a loss, if it is to fulfill the functions for which it was built. During this period, its income is commonly insufficient to meet the operating expenses, fixed charges, and fair returns. In order to create a business that will produce income sufficient to meet the legitimate expenses and a fair return, will require time, and involve direct expense and losses in meeting extraordinary expenses as well as necessary fixed and operating costs, and a temporary absence of profits.

The contingencies in the development of a successful business are among the most serious factors in any undertaking. Proper engineering advice will

anticipate the contingencies of construction, and experience will closely estimate costs of construction and operation. Even experience, however, can but poorly forecast the local idiosyncrasies on which the development of a business depends, unless a similar business has already been built up in the community. These facts are well known and recognized by financiers and investors and are reflected in higher rates demanded for bonds, preferred stock and other forms of securities. After the business has been developed, when the project has become a going concern and is earning fair returns on its actual value, a far different condition obtains. The property will then have proved its fitness for service and will have justified its creation, which gives investors a confidence that at once improves the financial prospects of the venture, increases its ability to finance extensions or to re-finance its loan on a lower basis, makes it a more conservative investment, and increases its value.

The history of any business development will substantiate these statements. As an example may be cited a modest hydro-electric development in the Middle West in which real money was invested in both common and preferred stock, and the cost of financing was, therefore, moderate. The bonded debt of this company was less than half the total investment, and the preferred stock issue was about half that of the common. This company completed its plant about 1912 and was obliged, in financing, to sell its bonds on about a 7% basis and its preferred stock on an 8% basis. By 1916, it had passed the development period, which cost its stockholders in expense and deferred income more than 15% of its total capital cost. In re-financing the project during 1916, in order to provide for plant extension, the company was able to retire its old securities and place its bonds and preferred stock at 5½% and 7%, respectively. This improvement in financial conditions immediately increased the value of the common stock by an amount greater than the development expense incurred.

In order to illustrate the principles involved and the methods that may be used in the study of a problem of this kind, a brief summary of the financial aspect of a hydro-electric development proposed in one of the Western States is given, as follows:

The cost of the hydro-electric development was estimated at $9 103 000, with $7 044 000 additional for the transmission system, making a total of $16 147 000 for the entire installation. As the State was contemplating the development of this project, and the bonds would be an obligation of the State, the interest on the capital to be invested would not exceed 5 per cent. On this basis, the actual cost of power, regardless of the total output generated and sold, was estimated at $1 412 050. The total annual output of the plant was estimated at 87 600 000 kw-hr. per annum which would result in a unit cost of 1.6 cents per kw-hr. when all the power was sold. As one of the main purposes of this plant was to further the industrial development of the State, it was proposed to sell the output at a price as low as possible, which was estimated at 1.87 cents per kw-hr. The generating costs in the various municipalities of the State varied from 1 cent to 15 cents per kw-hr., on account of the high cost of fuel. It was assumed that, within the first year after the construction of the plant and transmission line, a total load of

30 000 000 kw-hr. per annum could be obtained, and that the annual growth would be 10 per cent. The sale of 30 000 000 kw-hr. per annum on the basis of an average selling price of 1.87 cents per kw-hr. would be insufficient to pay interest and operating expenses, and the deficit would amount to $407 500 for the first year. With a 10% annual growth in the market, the deficit would decrease each year until the sale of power reached an annual amount of 53 000 000 kw-hr., when the loss would cease, and thereafter a profit, above interest and operating charges, would result. Table 2 shows in detail the estimated annual loss and gain, considering only interest and operation costs, with the 10% annual growth as estimated. From this table it will be noted that, if the plant had started with an annual output of 30 000 000 kw-hr. in 1919, and the output had increased uniformly at an annual rate of 10%, by 1924 the total loss (not considering interest on the deficit) would have amounted to $1 486 400, and that if interest on the losses were included, the total loss would have amounted (in 1925) to $1 812 168, not including depreciation. If depreciation is included on the assumed straight-line basis (of 100% depreciation in 38 years), the total loss would be $7 458 158, or about 46% on the cost of the plant. This amount is the cost of developing the market and is a part of what is ordinarily termed development expense. This is commonly represented by lost dividends in a privately owned plant and increases until the earnings reach a normal dividend-paying basis.

TABLE 2.—DEVELOPMENT EXPENSE BASED ON POSSIBLE INTEREST LOSSES DURING EARLY YEARS OF OPERATION; INCLUDING COST OF ENTIRE PLANT WITH PRIMARY AND SECONDARY TRANSMISSION SYSTEM. DEPRECIATION DEFERRED.

| Year. | Output in kilowatt-hours. | Cost of power, in cents. | Average sale price, in cents. | Difference of cost and sale price, in cents. | Loss. | Profit. | Accumulated loss, including 5% interest charge. | Accumulated profit, including 5% interest charge. |
|---|---|---|---|---|---|---|---|---|
| 1919 | 30 000 000 | 3.23 | 1.87 | —1.36 | $407 500 | .......... | $ 407 500 | .......... |
| 1920 | 33 000 000 | 2.94 | 1.87 | —1.07 | 353 000 | .......... | 780 875 | .......... |
| 1921 | 36 300 000 | 2.67 | 1.87 | —0.80 | 290 500 | .......... | 1 110 419 | .......... |
| 1922 | 39 930 000 | 2.43 | 1.87 | —0.56 | 223 500 | .......... | 1 389 440 | .......... |
| 1923 | 43 923 000 | 2.21 | 1.87 | —0.34 | 149 000 | .......... | 1 607 912 | .......... |
| 1924 | 48 315 300 | 2.00 | 1.87 | —0.13 | 62 900 | .......... | 1 751 207 | .......... |
| 1925 | 53 146 830 | 1.82 | 1.87 | +0.05 | .......... | $ 26 600 | 1 812 168 | .......... |
| 1926 | 58 461 513 | 1.66 | 1.87 | +0.21 | .......... | 122 799 | 1 780 075 | .......... |
| 1927 | 64 307 664 | 1.51 | 1.87 | +0.36 | .......... | 231 500 | 1 637 580 | .......... |
| 1928 | 70 738 430 | 1.37 | 1.87 | +0.50 | .......... | 353 500 | 1 365 959 | .......... |
| 1929 | 77 812 273 | 1.24 | 1.87 | +0.63 | .......... | 491 000 | 943 257 | .......... |
| 1930 | 85 593 500 | 1.13 | 1.87 | +0.74 | .......... | 634 000 | 356 420 | .......... |
| 1931 | 87 600 000 | 1.10 | 1.87 | +0.77 | .......... | 675 000 | .......... | .......... |
| Total.......... | | | | | $1 486 400 | $2 534 300 | .......... | .......... |
| Loss without interest.......... | | | | | .......... | 1 486 400 | .......... | .......... |
| Profit without interest.......... | | | | | .... .... | 1 047 900 | .......... | $300 759* |

*Profit including interest on early losses.

With the State plant, a profit (above interest and operating expenses) is shown for 1925, which increases annually until the capacity of the plant is reached in 1931, after which the annual profit would remain essentially stationary unless the sale price is changed.

Considering the entire thirteen years represented in Table 2, the gain (over operating expenses) exceeds the losses by about \$1 048 000, although if interest on the deficits is considered, this amount would be reduced to about \$300 000. This would be increased thereafter at the rate of about \$675 000 per annum until a normal depreciation fund is established after which the earnings could be applied to the liquidation of the capital cost.

The calculations for the following years are shown in Table 3. The interest losses would be recovered by the end of the thirteenth year (1931). The assumed depreciation would be recovered by the end of the twenty-fourth year, and if the total income above interest and operating expenses was applied to paying off the bonds, the capital cost would be paid at the end of the twenty-ninth year. For the remainder of the useful life of the plant, the savings above operating expense, interest, and depreciation would accumulate to (without interest on accumulation) \$13 266 000. Fig. 5 shows the theoretical financial operation of a contemplated hydro-electric development, giving estimated annual income, operating expenses, interest and depreciation, with all moneys applied to retirement of bonds until paid and ultimate earnings before final 100% depreciation. Fig. 6 illustrates the theoretical financial operation of a contemplated hydro-electric development, showing estimated capital invested, capital cost of developing market, retirement of bonds, and net earnings before ultimate final 100% depreciation, based on the annual financial showing of Fig. 5. It is evident that the depreciation used in this example is much too great and that the funds would not be used as outlined. Part of the funds would be expended in maintenance and replacements. The bonds would run for a greater length of time, and the plant, if properly constructed, operated, and maintained, would still be in a satisfactory operating condition at the end of thirty-eight years.

## TABLE 3.

| End of year. | Earnings. | Accumulated earnings. | Interest. at 5%. | Retirement fund. | Accumulated depreciation. | Accumulated net profits. |
|---|---|---|---|---|---|---|
| 1931 | \$ 675 000 | \$ 300 000 | ........ | \$ 300 000 | \$ 5 645 990 | .......... |
| 1932 | " | 975 000 | \$ 15 000 | 990 000 | 6 080 220 | .......... |
| 1933 | " | 1 665 000 | 49 500 | 1 714 500 | 6 514 450 | .......... |
| 1934 | " | 2 389 500 | 85 725 | 2 475 225 | 6 948 680 | .......... |
| 1935 | " | 3 150 225 | 123 761 | 3 273 986 | 7 382 910 | .......... |
| 1936 | " | 3 948 986 | 163 790 | 4 112 776 | 7 817 140 | .......... |
| 1937 | " | 4 787 776 | 205 638 | 4 993 414 | 8 251 370 | .......... |
| 1938 | " | 5 668 414 | 249 670 | 5 918 084 | 8 685 600 | .......... |
| 1939 | " | 6 593 084 | 295 904 | 6 888 988 | 9 119 830 | .......... |
| 1940 | " | 7 563 988 | 344 449 | 7 908 437 | 9 554 060 | .......... |
| 1941 | " | 8 583 437 | 395 421 | 8 978 858 | 9 988 290 | .......... |
| 1942 | " | 9 653 858 | 449 392 | 10 103 250 | 10 422 520 | .......... |
| 1943 | " | 10 778 250 | 505 162 | 11 283 412 | 10 856 750 | .......... |
| 1944 | " | 11 958 412 | 564 170 | 12 522 582 | 11 290 980 | .......... |
| 1945 | " | 13 197 582 | 626 129 | 13 823 711 | 11 725 200 | .......... |
| 1946 | " | 14 498 711 | 691 185 | 15 159 896 | 12 159 400 | .......... |
| 1947 | " | 15 864 896 | 759 494 | 16 624 890 | 12 593 700 | .......... |
| 1948 | 1 474 000 | .......... | | | 13 027 900 | \$ 1 474 000 |
| 1949 | .......... | .......... | ........ | .......... | 13 462 100 | 2 948 000 |
| 1950 | .......... | .......... | ........ | .......... | 13 896 400 | 4 422 000 |
| 1951 | .......... | .......... | ........ | .......... | 14 330 600 | 5 896 000 |
| 1952 | .......... | .......... | ........ | .......... | 14 764 800 | 7 370 000 |
| 1953 | .......... | .......... | ........ | .......... | 15 199 000 | 8 844 000 |
| 1954 | .......... | .......... | ........ | .......... | 15 633 300 | 10 318 000 |
| 1955 | .......... | .......... | ........ | .......... | 16 067 500 | 11 792 000 |
| 1956 | .......... | .......... | ........ | .......... | 16 501 800 | 13 266 000 |

A further examination of Table 3 will show that if the market could be developed, so that the demand at the time of completion of the plant would reach 53 000 000 kw-hr. per annum, the income received would be sufficient to meet the interest and operating expenses and would be sufficient by 1929 to meet the annual depreciation charges as well.

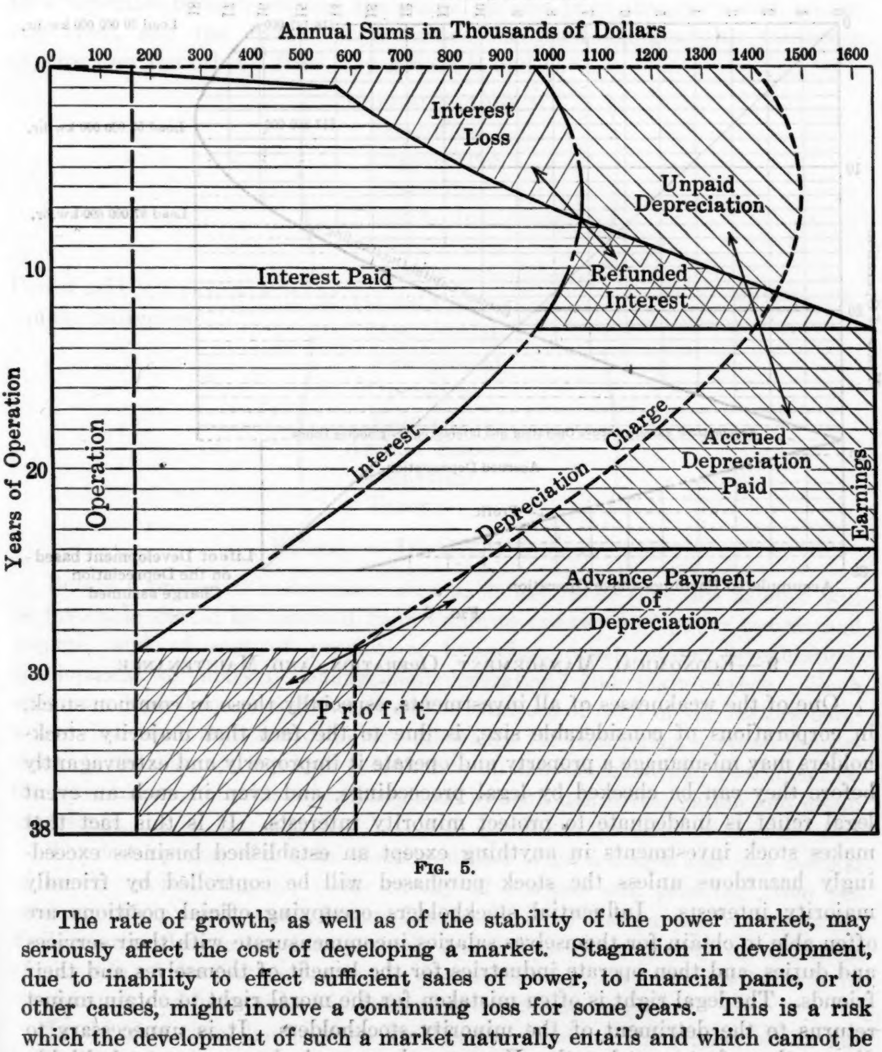



FIG. 5.

The rate of growth, as well as of the stability of the power market, may seriously affect the cost of developing a market. Stagnation in development, due to inability to effect sufficient sales of power, to financial panic, or to other causes, might involve a continuing loss for some years. This is a risk which the development of such a market naturally entails and which cannot be obviated.

It is evident, therefore, that the success of a contemplated development depends not only on the original cost of installation, but also on the growth and development of a power market. It is not assumed that this showing would be attractive to a State, and much less to a private corporation, but it it believed that the principles involved and correctly applied furnish an idea

of how such studies may be made. Any plant can then be studied on such basis as may seem desirable. The basis should be in general conservative, even to the extent of being severe. A project should show a considerable factor of safety to take care of future conditions which cannot be foreseen.

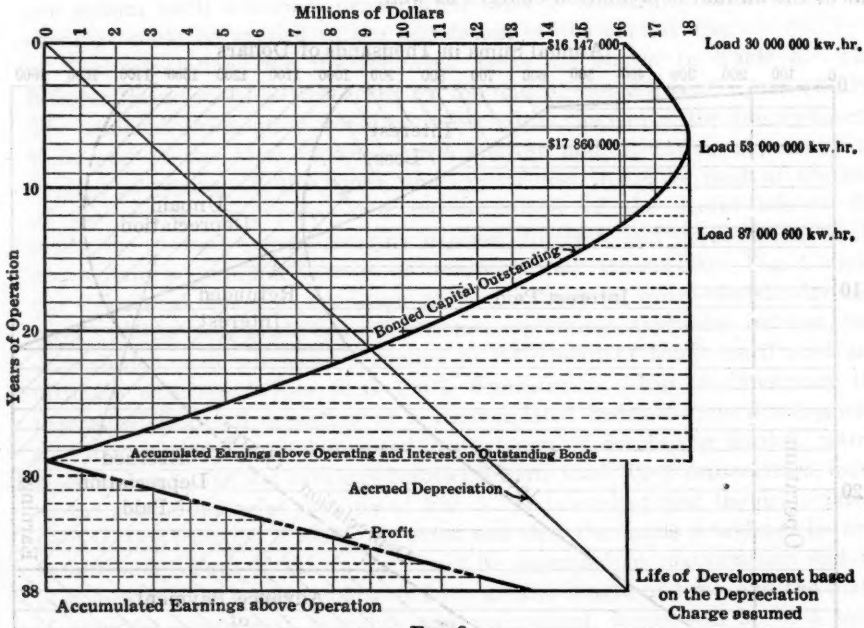



FIG. 6.

## 9.—ECONOMICAL MANAGEMENT, OPERATION, AND MAINTENANCE

One of the weaknesses of all investments, especially those in common stock, in corporations of considerable size, is due to the fact that majority stockholders may mismanage a property and operate it improperly and extravagantly before they can be checked by legal proceedings, and even in such an event legal relief is inadequate to protect minority interests. It is this fact that makes stock investments in anything except an established business exceedingly hazardous unless the stock purchased will be controlled by friendly majority interests. Influential stockholders occupying official positions are often able to obtain for themselves salaries incommensurate with their services and duties, and then operate industries for the benefit of themselves and their friends. The legal right is often mistaken for the moral right to obtain unjust returns to the detriment of the minority stockholders. It is unnecessary to discuss these factors at length. Honest and economical management is highly essential to the success of any project.

The operators of a plant should be well trained and experienced, adequate in number, but limited by the best interests of the plant. Attention should be given to every possible source of waste, and maintenance must receive adequate attention. These matters can result only from honest, intelligent, and

experienced management, which is sometimes a considerable hazard in new business ventures.

### 10.—Accurate Estimates

The economic expediency of any hydro-electric development will be justified by a favorable balance between the annual income from the power sold, on the one hand, and the annual expenditures for fixed charges plus operating expenses on the other. (See Appendices B and C.)

| | | |
|---|---|---|
| Power sold × Price = Annual income *vs.*........ | Interest on Cost of:<br>Promotion<br>Engineering<br>Financing<br>Rights and Property<br>Construction<br>Market Development<br>+ Annual Cost of:<br>Depreciation<br>Management<br>Operation<br>Maintenance<br>+ Annual Expense of:<br>Insurance<br>Taxes | = Total annual expense |

Balance = Dividends or losses

Before a project is undertaken, reasonable assurance that this balance will be favorable should be obtained from accurate estimates of power output and income, and of cost of promotion, financing, construction, depreciation, development of market, management, operation, and maintenance. Such estimates depend on correct and complete analyses of each of the factors heretofore discussed, and should include allowance for contingent expense proportional to the hazards involved. Optimism on the one hand and pessimism on the other must be eliminated and replaced by thorough investigation and broad experience.

The development of a hydro-electric project is not a simple and easy method of surely capitalizing the waste energy of streams and profiting from the returns. It involves many factors besides those of design and construction. Failure or neglect to consider properly and determine accurately the influence of any one of the factors previously discussed, may create serious hazards in the venture, and has resulted in failures more or less complete.

In the past there has been an idea in the minds of the public, which has been shared by many investors and by many engineers, that as undeveloped water power is energy going to waste, the development of such power must always result in large and profitable returns. On this hypothesis, and with little or no additional consideration, some hydro-electric projects have been conceived, designed, and constructed, and have resulted either in utter failure or in only partial fulfillment of the hopes that inspired their promotion. With the

increasing demand for power, coupled with the popular conception that water powers were always exceedingly profitable, and that by means of such developments the waste energy of a stream could advantageously be turned into dividends, investors eagerly sought hydro-electric investments. The results have not always been satisfactory, and a considerable number of large and small developments might be cited, which have resulted in serious financial catastrophes and failures to the original investors in the enterprises.

Foreclosures and sales of water-power properties have been only too common and, at present, there are some developments operating under conditions which barely permit of their survival. In one case within the State of Wisconsin, after foreclosure, the investors in the bonds of a water-power company realized less than 5% of their par value. In another case, the plant was definitely abandoned and dismantled (see Fig. 7). It is evident that such projects were ill-advised and should never have been undertaken or, if attempted, undertaken on a more conservative basis. In many of the failures which might be cited, it is evident that most of the difficulties in which hydro-electric developments have been involved, have been due to lack of foresight, as to the factors necessarily involved in a successful development, or to lack of a thorough investigation and determination of the character of these factors and their influence on the project. Most if not all these failures could probably have been avoided by intelligent and thorough investigation which would have led either to the abandonment of the project or to a more conservative development.

Plans for large and important structures are rarely devised, that do not require modification during construction. Unless this fact is appreciated by the designer, and liberally allowed for, the estimate of cost will always be inadequate to complete the project.

The hazards of construction increase with the difficulties. When a structure is built in and across a river, the work of construction is subject to certain hazards which cannot fully be foreseen, but which must be recognized and provided for in the cost estimates; there are contingencies of foundation which require careful investigation, and of flood which experience has shown to be very important. The investment is not easily determined, and the ultimate cost is sometimes greater than inexperience would deem possible.

In a recent large water-power development which was under advisement for a number of years, and which was supposed to be thoroughly considered and carefully planned both as to its design and the methods of construction, the cost of the finished structures exceeded the estimate by 33 per cent. In another case, with which less care was used, on account of the belief in the great profits from hydro-electric developments, the cost of the complete development was more than three times the original estimated cost; and since its foreclosure and sale, the plant has not been found to be a profitable investment, even on the basis of the purchase price, which was 20% of its actual cost.

The unexpected extra costs of such developments due to unforeseen delays are often serious. The interest on bonds must be met semi-annually or annually

FIG. 7.—VIEW OF DISMANTLED AND ABANDONED HYDRO-ELECTRIC DEVELOPMENT IN WISCONSIN.

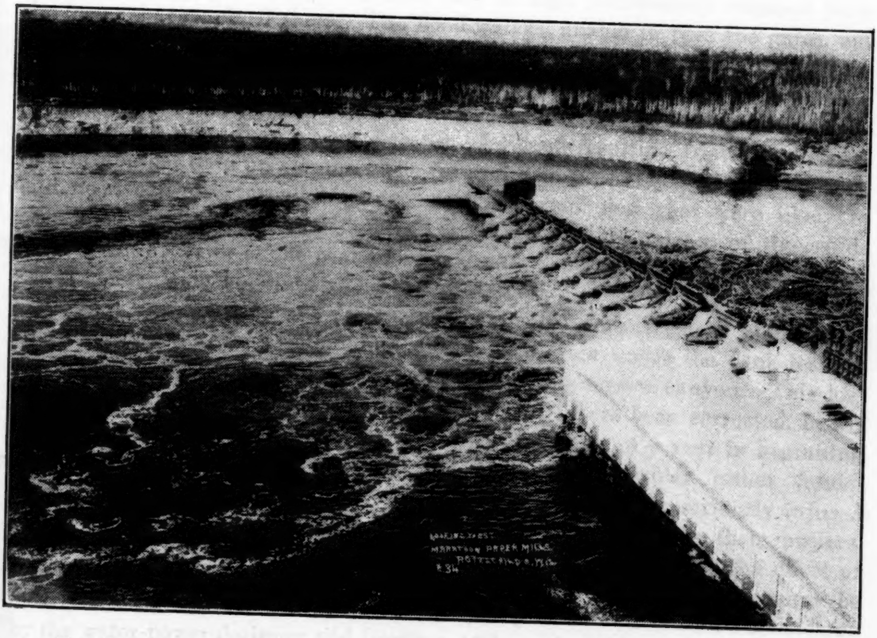


FIG. 8.—VIEW OF DAMAGE TO HYDRO-ELECTRIC DEVELOPMENT DUE TO LOG-JAM.

FIG. 7.—VIEW OF DISMANTLED AND ABANDONED HYDRO-ELECTRIC DEVELOPMENT IN WISCONSIN.

FIG. 8.—VIEW OF DAMAGE TO HYDRO-ELECTRIC DEVELOPMENT DUE TO [illegible]

from their date of issue; hence, interest during construction is an important item in the cost of development of any industry, and an item which is particularly uncertain in water-power development. In a recent hydro-electric development, a flood (the largest that had occurred on the river within the limited period of record) not only caused a large loss to the work under construction, but was followed by continuous and unusual high water for most of the year following, so that not more than ninety working days were available within the year. In the same project, an ice jam in the spring carried away the trestle and falseworks. These misfortunes caused a delay of more than a year, with an extra interest cost of many thousands of dollars.

Such hazards obtain more in all classes of hydraulic work than in almost any other kind of development, except mining. Although care and experience with hydro-electric projects will result in more careful investigation and more liberal estimates so as to provide for contingencies that are likely to occur, still the contingencies exist and the necessary investment, in the future as in the past, will frequently be under-estimated. The costs of construction will often require greater investments than the optimistic promoters will think possible, even when money is judiciously and cautiously expended.

Even after a development is completed, all contingencies are not removed. Not many years ago, a flood in one of the rivers of Wisconsin caused a loss of several hundred thousand dollars to a single development. This loss resulted from an extraordinary and uncommon condition. The gates of the plant were clogged by an unexpected rush of logs, accidentally released from the boom that confined them. The flood-waters, unable to pass the gates, cut their way around the structure, made necessary a large expense for new construction, and put the plant out of service for more than a year. (Fig. 8.) At another plant development, a dam was seriously injured, and the plant was out of service for more than a year, because of a flood resulting from the destruction of another dam up stream. The injured dam possessed sufficient strength and flood capacity for all contingencies of normal flow that were likely to occur, yet the destruction of another structure and the release of thousands of acre-feet of water at the peak of a maximum flood, resulted in large expense for reconstruction and in serious loss of service.

Losses through defective designs are common. In a Western plant about to be started, the whole foundation was washed from under the dam, leaving the structure suspended from the rock walls of the narrow canyon. This loss was due to the defective design which could easily have been corrected, but it cost the company many thousands of dollars and delay of a year in beginning operations. Numerous instances could be given in which either fundamental defects or extraordinary conditions have destroyed or seriously injured dams and hydro-electric plants and caused serious losses to their owners. Knowledge of the possible activities of natural forces is limited; the effect of the possible combination of the known and unknown factors can never be clearly seen, yet these contingencies are ever present and must be considered by the water-power designer and investor, and contingent costs must be allowed in estimates in proportion to the hazards involved.

In estimating power, income, and costs in hydro-electric developments, it is evident, therefore, that a thorough study of each factor is essential to a correct solution of the problem.

Before closing it seems pertinent to discuss briefly a still broader field of the economics of hydro-electric developments; that is, the relation of such development to the public, the State, and the Nation. Much has been publicly said and written adverse to the development of these powers by private interests and favorable to State or Federal ownership and development. In the first place, the great value of the majority of the water powers of the Nation is potential only. More than 50% of these powers could not profitably be developed, and they are, therefore, valueless except on the speculative basis of future possibilities.

The development of these powers adds greatly to the tax income of the State and the Nation. Their utilization entails extensive developments in industry, in opportunity, in population, and in the value of taxable property. These powers decrease the demand for coal from 5 to 10 tons per annum per horse-power developed, which is equivalent to a saving of from one-sixth to one-third of a carload of coal per annum per horse-power. This transporting capacity can then be devoted to the service of an equivalent amount of manufactured or agricultural products. The development of these powers entails an equivalent development in industry and an equivalent satisfaction of some service or convenience to the people of the Nation. The State in which they are developed generally receives in taxes about 2% of the cost of the development per annum which, within the limits of a Federal franchise, amounts to as much as the capital cost of the development, and the indirect income from industrial development will be fully equal and probably greater. The benefit to the Nation, the State, and the community is such that, regardless of the trend of future ownership, the benefits derived by the public warrant every reasonable encouragement by the State and Federal Governments.

Although these developments are directly undertaken for the possible profits to be derived from their construction and operation, the public receives, in general, no less benefit from their development than the corporations which develop them; and, in some cases, it can be shown that within ten years of the construction of such developments, their customers have received in reduced power charges more than the capital costs of the entire development. Direct benefit to the people is assured from the very nature of the commercial conditions of the sale of such power, as has already been pointed out, and even under non-competitive or monopolistic conditions, their benefit is assured through the State control of power rates.

It is true that the public often anticipates greater saving through rate reduction than commonly is possible. Distribution is commonly about 60% or more of the cost of delivered electric power, therefore, any cheapening in power generation will not effect more than 40% of the cost to the consumer.

In supplying power to rural lines, the distribution cost is commonly more than 90% of the total cost; and even if the power cost nothing to generate, the cost of power from such lines will necessarily be high, compared with the price at which it can be furnished in cities and especially to large consumers.

Incidentally, it may be stated that the popular and newspaper conception of interconnection of plants and "super-power" developments is often erroneous. The mere interconnection of plants may not of itself effect economies or in any way warrant a reduction in power rates. Such interconnections may be, and actually are, warranted in certain cases, and frequently provide greater security and stability in the power supply of any community by affording an additional source of supply in case of local accident, or even due to local low-water conditions, and, broadly speaking, such interconnection may bring about certain economies that may ultimately result in rate reduction. Such connections, however, cannot produce radical reduction in rates, or bring to small customers on rural lines the advantage of the energy rates of large customers, as is sometimes apparently anticipated.

Those who are developing the resources of the Nation under reasonable conditions are not enemies of the public, and the writer believes that he who causes two blades of grass to grow where one grew before is no less worthy of credit if he garners his harvest and sells it for his own profit.

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## APPENDIX A

### Factors of Economic Expediency in the Consideration of Proposed Hydro-Electric Developments

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The success of hydro-electric developments will depend on the following factors:

1.—The acquisition of suitable legal rights:
- (*a*) To a franchise or legal right to build and operate.
- (*b*) To the water supply.
- (*c*) To the site for dam and power station.
- (*d*) To the flowage, including pondage and storage.
- (*e*) To the rights of way for ingress, egress, transmission lines, canals, pipe lines, etc.

2.—The command of a satisfactory market:
- (*a*) Within reasonable transmission distance.
- (*b*) Of sufficient size to utilize a large proportion of the power.
- (*c*) Of a character to permit of an economic development (high load factor).
- (*d*) At a price sufficient to warrant the investment.
- (*e*) Of a stability that will warrant a permanent or definitely limited investment in real estate, water right, plant, transmission line, etc.
- (*f*) Reasonably free from likelihood of damaging competition.

3.—Reasonably suitable physical (topographical and geological) conditions:
- (*a*) For a dam site and sites for construction of power-house and appurtenances.
- (*b*) For the development and maintenance of a suitable head.
- (*c*) For stable and permanent structures at reasonable expense.

(*d*) For impounding water for pondage and storage, with only moderate losses from percolation and evaporation.

(*e*) For economical conduct of the work:

For transportation of materials and machinery.

For securing labor, teams, and construction equipment.

For securing bidders on the work.

4.—Favorable hydrological conditions:

(*a*) To provide a stream flow not subject to extreme variations of discharge; that is fairly well maintained at low-water seasons, and free from extreme floods.

(*b*) To require only a small proportion of auxiliary power.

(*c*) To provide the amount of power necessary for the market, with the use of such other water or auxiliary power as is economically available.

5.—Proper design:

(*a*) Location selected must be the most favorable:

To afford suitable foundation for dam, power-house, and other work.

To afford ingress and egress and reasonable transportation facilities.

To afford sufficient pondage to equalize the flow and permit its utilization on the load factor under which it will operate up to the point of economical development.

(*b*) The dam:

Must have suitable foundation.

Must have ample capacity for maximum flood.

Should have gates to keep low-water head at maximum while providing ample capacity (with gates open) to keep head and back-waters at their designed limits.

Must be designed for strength and stability; also, to resist shocks of floating ice or débris and for ice pressure.

Must have base and ends essentially tight against seepage.

Must be designed to prevent overflow around ends.

Must be designed to pass ice, débris, and logs.

Must be designed with suitable provision against erosion down stream from the structure.

Should have such logways, fishways, mud valves, sluice-gates, or head-gates, as may be necessary.

Should be long enough for passing floods and short enough to avoid high cost.

(*c*) Head-race (where needed):

Should have economical section to prevent loss of head (and energy) under normal flow, at peak load, and under ice conditions.

Should be provided with a forebay just above the plant for small storage to permit change of load without serious loss of head.

Usually with head-gates or stop-logs openings.

Usually with booms to prevent ingress of floating débris.

Lined to prevent seepage and to reduce friction.

Stable to reduce cost of maintenance and repairs.

(*d*) Pipe penstocks or tunnels (where needed):

Economical section.

Proper head-gates and protection from drift.

Proper inlet gates for filling penstocks slowly and equalizing pressure on head-gates before opening.

Air inlet to prevent collapse when being emptied.
Stand-pipe or surge tank to prevent shocks and stabilize running condition of turbines and generators.
Proper entrance to turbine chamber.

(*e*) Power-house should be designed:

As to substructure:

On safe and stable foundation.
With trash-racks to keep débris from turbines, but with spacing and section not to interfere with flow and suitable for convenient and easy raking.
With trash-gates to permit disposal of trash.
With head-gates to cut off supply from turbines and inlet gates to fill penstocks and equalize water pressure.
With suitably shaped inlet passages to turbines.
With suitable draft-tubes and discharge sections to reduce velocity losses.

As to superstructure and sub-stations:

With ample room for equipment.
Arranged with regard to safety in operation.
Arranged to facilitate operation (so that the right way is the most convenient and easiest way to do).
For future enlargement or extension (if probable).
Fire-proof, with ample light and suitable ventilation.

(*f*) Tail-race:

With economic section to facilitate discharge and economize head (see also "Head-Race").

(*g*) Equipment:

General:

Gate-operating machinery for ease and rapidity of handling gates.
Cranes for economic erection and maintenance of machines.
Recording gauges to give suitable records.

Turbines:

Of an economical type.
Well-designed and constructed.
Equipped for renewable parts.
Not too large for economical operation.
With fly-wheels, if necessary for proper regulation.
With sensitive governors.

Electrical equipment, generators, exciters, switchboard, protective devices, automatic or remote control, lightning arresters, transformers, oil purifiers, switching, and metering equipment.

(*h*) Transmission line:

Right of way easy of access for construction and maintenance, ample in width to protect line from falling timber, encroachments, and fires.
Safety to public by warnings, height of lines, and protection under lines at highways, railroads, and other crossings.
Economical voltage for distance.
Conductors large enough for proper regulation and low-power loss.
Adequate insulation and lightning protection.
Mechanical strength to withstand wind and sleet.

6.—Economical construction:

(a) Proper supervision, inspection, testing, etc.

(b) Foresight and provision for contingencies.

(c) Judicious contracts at reasonable prices, or construction by day's work under experienced economic supervision.

(d) Judicious purchase of materials, machinery, equipment, and supplies.

(e) Curtailment of undue salaries and general expenses.

7.—Financing:

Although this item is included under "Accurate Estimates", it also requires special consideration, as many developments are seriously handicapped by under or over-financing, or by too great cost of financing.

8.—Rapid and economical development of business:

Although this item is considered under "Command of a Satisfactory Power Market", attention is especially called to the fact that the amount of power which can be sold immediately, and the price which can be obtained, is frequently over-estimated; also that the time required for developing a profitable market often requires a larger extra investment in actual cost, interest on bonded debt and deferred dividends.

9.—Economical management, operation, and maintenance:

A development otherwise satisfactory may be seriously handicapped through unwise and extravagant management. Honest and economical management is highly essential to economic success. Experienced and trustworthy operation and maintenance is also essential. Waste, with consequent loss of output, may arise from neglect in operation, such as racks clogged with débris, leaking turbine gates or runners, and the like. Many such sources of loss and waste are possible and can be obviated by wise and intelligent work.

10.—Accurate estimates of power output and income, and cost of promotion, financing, construction, depreciation, development of a market, management, operation, and maintenance:

The final criterion of economic development of any hydro-electric power is a favorable balance between annual income and annual expense. An accurate forecast of all the items which go to make up this income and these expenditures is, perhaps, the most difficult in an investigation, and the balance should be sufficiently favorable to cover all hazards that are likely to occur.

## APPENDIX B

### Capital Costs of Hydro-Electric Developments

1.—Preliminary Investigation Expenses:

(a) Legal investigations of water rights, land rights, transmission rights of way, etc.

(b) Surveys, geological and hydrological investigations.

(c) Foundation conditions, borings, etc.

(d) Census of power market and determinations of stability and sale price of power.

(e) Preliminary designs and cost estimates.

2.—Promotion and Organization Expense:
- (*a*) Promotion costs and profits:
- (*b*) Legal expenses of organization.
- (*c*) Cost of obtaining franchise.
- (*d*) General expense of organization.
- (*e*) Expense of financing and placing stocks and bonds or other securities.

3.—Real Estate and Water-Right Expenses:
- (*a*) Sites for dam and power-house.
- (*b*) Flowage and water rights.
- (*c*) Rights of way for canals, roads, and transmission lines.
- (*d*) Sites for auxiliary plant and sub-station.

4.—Cost of Plans:
- (*a*) Final and complete surveys and investigations of site, flowage, and location of canals, roads, and transmission lines.
- (*b*) Complete plans, specifications, and contracts.
- (*c*) Final estimates of cost of complete plant and equipment.

5.—Construction Costs:
- (*a*) Cost of clearing lands, including flowage.
- (*b*) Cost of plant with complete equipment, including materials, labor, construction equipment, and plant supplies.
- (*c*) Cost of operators' houses, if required.
- (*d*) Contingent cost, with due regard to each feature of the development.
- (*e*) Cost of engineering, supervision, inspection, tests, administration expense, including salaries, insurance, office equipment, and office supplies.

6.—General Costs:
- (*a*) Insurance and taxes.
- (*b*) Interest during construction.
- (*c*) Interest during development and development expenses.
- (*d*) Stores and supplies.
- (*e*) Working capital.
- (*f*) General administration expenses.
- (*g*) Expense of financing (if not included in promotion expenses).

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## APPENDIX C

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### Annual Expenses

1.—Salaries and Wages:

Officers, bookkeepers, and clerks.
Management and supervision.
Power salesmen and adjusters.
Operators, linemen, etc.

2.—Supplies:

Fuel and oil.

3.—Maintenance:

Repairs and replacements.

4.—Interest:

On bonds, notes, and floating debt.

5.—Discount on:
Stock and bonds, when not made a capital charge, must be provided for by an annual charge.

6.—Development Cost:
When not provided for as a capital charge, this must be absorbed as an operating expense.

7.—Replacement Reserve:
To provide for unusual damages to plant and replacement of major units of plant.

8.—Sinking Fund:
To retire values when values may be destroyed under a limited franchise, and to reimburse the investor on the expiration of the franchise.

## APPENDIX D

### Calculation of Net Revenue (Gross Revenue Less Steam Power Cost) for Peninsular Power Company

By F. E. Turneaure, M. Am. Soc. C. E.

The lower half of the average duration curve for the Twin Falls Plant is closely expressed by the equation:

$$y = 1\,800 + \frac{t}{6}\,(2\,200); \text{ or } t = \frac{6}{2\,200}\,(y - 1\,800)$$

in which,

$y$ = horse-power available from hydraulic plant; and

$t$ = time, in months, during which the stream flow is below that necessary to deliver $y$ h.p.

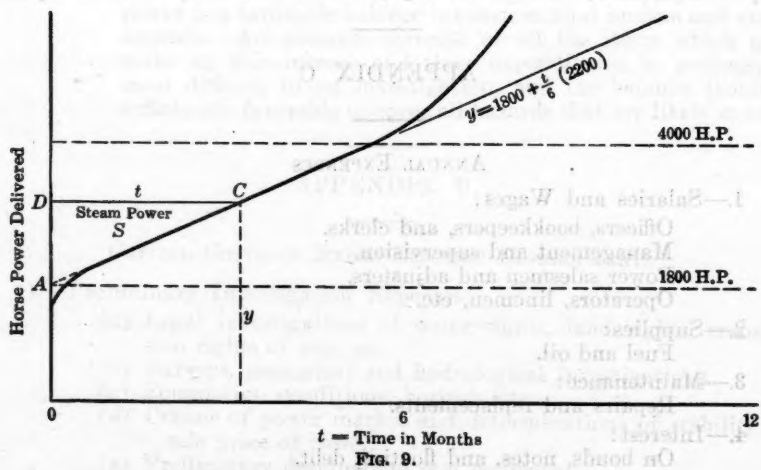


Fig. 9.

The amount of power necessary to be generated by steam in the average year, in order to supply $y$ h.p. continuously, will then be the area, $A\,C\,D$. Expressed in horse-power-months, this equation is:

$$S = \frac{1}{2}(y - 1\,800)\,t = \frac{3}{2\,200}(y - 1\,800)^2$$

If $r$ = average gross income per horse-power-month and $c$ = cost of steam power per horse-power-month, the net yearly revenue for $y$ continuous horse-power will be:

$$R = 12\,r\,y - \frac{3}{2\,200}\,c\,(y - 1\,800)^2$$

The average monthly revenue per horse-power is almost exactly $6.00 (= 1.1 cent per kw-hr.), or $r$ = $6.00.

For various values of $c$, the amount of net revenue is then, as follows:

$c = 1\frac{3}{4}$ cents per kw-hr. = $9.65 per horse-power-month.

$$R = 72\,y - 0.013\,(y - 1\,800)^2$$

$c = 2$ cents per kw-hr. = $11.00 per horse-power-month.

$$R = 72\,y - 0.015\,(y - 1\,800)^2$$

$c = 2\frac{1}{4}$ cents per kw-hr. = $12.40 per horse-power-month

$$R = 72\,y - 0.017\,(y - 1800)^2$$

$c = 2\frac{1}{2}$ cents per kw-hr. = $13.70 per horse-power-month.

$$R = 72\,y - 0.019\,(y - 1\,800)^2$$

For the maximum value of $R$, $y = 1\,800 + \dfrac{72}{2 \times 0.013}$, etc.:

For $c = 1\frac{3}{4}$ cents $y = 4\,500$ h.p.
$c = 2$ cents $y = 4\,200$ h.p.
$c = 2\frac{1}{4}$ cents $y = 3\,900$ h.p.
$c = 2\frac{1}{2}$ cents $y = 3\,700$ h.p.

The capacity of the Twin Falls and the Brule Plants may be placed at about 3 900 h.p. and the steam plant will have sufficient capacity to maintain this output at low water. The capacity is, therefore, not sufficient to secure the maximum revenue when $c$ is below $2\frac{1}{2}$ cents per kw-hr.

The net revenue for $y$ = 3 900 h.p., for various values of $c$ is, as follows:

For $c = 1\frac{3}{4}$ cents $R$ = $223 600
$c = 2$ cents $R$ = 214 800
$c = 2\frac{1}{4}$ cents $R$ = 206 000
$c = 2\frac{1}{2}$ cents $R$ = 197 200

For $c = 2\frac{1}{2}$ cents and $y$ = 3 700 h.p. (the most economical value), $R$ = $197 800, only slightly larger than for $y$ = 3 900.

If the capacity was sufficient, the value of $R$, for $c = 1\frac{3}{4}$ cents and $y$ = 4 500, would be $229 000, a difference of only $5 400 from the previously given value of $223 600 for $y$ = 3 900 h.p.

With coal at $10 per ton, the cost of steam power will be about $2\frac{1}{4}$ cents per kw-hr., and the most profitable output would be about 3 900 h.p., or 7 800 h.p. from the two plants with a net revenue of about $412 000. Adding

the Pine River Development at 60% of each of the others, brings the most profitable output to 3 900 × 2.6 = 10 200 h.p. and the net revenue to $535 000.

To deliver this amount of power would require further additions to the steam plant. Without such additions, the capacity will be about 8 800 h.p., taking the low-water capacity of the Pine River at 1 000 h.p. Distributing this load proportionately will call for 3 400 h.p. from Twin Falls, giving a net income ($c = 2\frac{1}{4}$ cents) of $201 300, or for the three plants of $524 000, or only $11 000 less than the maximum to be secured by additional steam. The additional steam capacity to supplement the Pine River development would be about 1 400 h.p., costing at least $125 000. Fixed charges on this would be about 11%, or $14 000, which shows that, at the present rates for current and for coal, there would be no profit in developing the capacity of Pine River beyond 1 000 h.p.

Query: Are the rates for power what they should be in the interests of "conservation"?

CALCULATION FOR MINIMUM YEAR (ENVELOPE CURVE FOR 1903-1918 INCLUSIVE)

The same method of analysis applied to the year composed of the lowest points for the years 1903-1918, gives the following value for net revenue:

$$R = 72\,y - \frac{3}{1\,100}\,c\,(y - 1\,600)^2$$

with $c = 2\frac{1}{4}$ cents per kw-hr., the net revenue from the two plants would be as follows:

For a total of 5 000 h.p. (the present load) $R = \$306\,000$
" " " " 6 000 h.p. " " " $R = 300\,000$

The most favorable load would be 5 300 h.p. A net revenue of $300 000 would leave about $70 000 for common stock. For the three plants, the net revenue would be:

For a total of 5 000 h.p. $R = \$377\,000$
" " " " 6 000 h.p. $R = 393\,000$

The most favorable load would be 6 900 h.p. and would produce a net revenue of $400 000. For the three plants, the fixed charges, operating expenses, and dividends on preferred stock would require about $300 000 per year.

# RESEARCHES ON THE STRUCTURAL DESIGN OF HIGHWAYS BY THE UNITED STATES BUREAU OF PUBLIC ROADS*

By A. T. Goldbeck,† Assoc. M. Am. Soc. C. E.

## Synopsis

The United States Bureau of Public Roads, in connection with its extensive work on Federal Aid Roads, has been authorized to do considerable research involving various phases of highway design and construction. This work has been carried on in different localities and with several co-operative agencies, but largely from the main office and at the Experimental Farm in Arlington, Va. This paper treats only those phases of the research concerned with the structural features of highway design, considered under (1) researches on the sub-grade; (2) loads on pavements; (3) stresses in concrete pavements; (4) Pittsburg, Calif., test road; (5) studies in materials; and (6) miscellaneous investigations. In all cases, the scope and procedure of the test project is given, results are described, and conclusions drawn. Although certain phases have yielded conclusive results, the work on highway research, as a whole, is far from complete; this paper simply aims to acquaint engineers with some of the accomplishments to date and to stimulate further development.

## Introduction

It has been estimated that the Federal Aid Highway System when completed, will comprise at least 180 000 miles of road; this will be less than 7% of the road mileage of the United States. At the present time, approximately 45 000 miles of the Federal Aid System are completed or under construction so that the uncompleted mileage of roads necessary to serve adequately present and future traffic reaches astounding figures. Construction of such magnitude requires the expenditure of millions of dollars and the complete investigation of both economics and technical features. Not all of the construction will be high-class pavements; much will be of a secondary nature carrying so little traffic that large expenditures on them would be economically unjustifiable. The main routes, however, and many secondary routes will carry a sufficient volume of travel to warrant paving them with the highest types of surfaces.

The proper adjustment of the road type to local conditions is, of course, a problem involving many considerations, for, after all, the road is merely one of the elements forming a transportation system, which should function as a whole with greatest economy. Full recognition is made, therefore, of the fact

* Presented at the meeting of the Highway Division, January 17, 1924.

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that the final design and choice of road type are governed by factors that must be given proper weight when considered in combination with another important factor, the ability of the roadway to carry traffic loads. The present discussion includes only those investigations which have been undertaken to shed light on the principles involved in the structural design of roads.

### Fundamentals Involved in the Structural Design of Roads

A roadway considered from the structural standpoint includes not only the surfacing material, but the underlying sub-grade, and also the shoulders and drainage structures. A wide variety of surfacings are used; some are of a more or less rigid and elastic nature, such as concrete; others have less rigidity and are at times quite plastic, such as the bituminous types; still others have practically no resistance to bending, but are highly resistant to heavy loads when given proper sub-grade support, such as the macadam, gravel, and sand-clay types of pavements.

The design of any engineering structure involves: (1) correct evaluation of the forces or agents which that structure must resist; and (2), the correct proportioning of that structure to resist those forces economically. Due recognition, of course, must be given to the choice of such materials as will be sufficiently resistant under the attacks of the chemical or physical agents. It is difficult to conceive of any engineering structure that is called on to withstand a more complex set of influences than a road surfacing.

The different types of pavements are constructed over sub-grades ranging from swamp lands and plastic clays to hardpan, compacted gravel, and even solid rock. Obviously, the influence of the sub-grade on pavement design has a very important place in highway research. Then, too, the presence of moisture in these different soils is continuously changing their characteristics, the degree of change depending on the nature of the soil. Again, the moisture content affects the volume in some cases as much as 50% of the dry contents. The influence of this phenomenon is apparent when it is considered that vertical motion of the road surface must accompany these changes in volume and that this motion varies between the sides and the center of the road. Frost action in the sub-grade also has a decided influence on the road surface in producing non-uniform vertical motion and resulting in deterioration of improperly designed pavements.

The effect of natural forces on the road surface itself must also be considered, as, for illustration, (*a*) the humping or "blowing up" of rigid pavements due to the expanding action of temperature and moisture; (*b*) the action of frost and low temperatures in producing horizontal movement, brittleness, or scaling of the different surfaces; (*c*) the disintegrating or softening action of moisture on certain types of materials; and (*d*) the variety of forces exerted by traffic, sometimes static but generally dynamic.

The element of fatigue of certain types of road surfacings enters still further to complicate the problem, for the loads on pavements are repeated millions of times during their life. It might seem to be almost impossible to arrive at a rational basis for road design when all these disturbing factors are known to exist, and possibly an exact application of any theory may be rendered

very difficult. Nevertheless, it is felt that a long step forward has been made in the direction of more correct knowledge. The results lend encouragement to the belief that the future design of roads will be placed on a much more rational and scientific basis than it is at present.

## Researches on the Sub-Grade

The character of support offered is always a most important matter in connection with the design of any structure, but most emphatically so in the case of a road surfacing. There are a number of ways in which the type of sub-grade material affects the ability of a road surface properly to support the traffic. The very finely divided plastic soils in the presence of moisture may absorb a high percentage of water by capillarity. They then become extremely soft and are easily deformed under the pressure.

The action of frost in causing heaving, due to the swelling of the sub-grade, is likewise well known. The amount of movement is influenced by the type of soil, because some soils are capable of retaining high percentages of moisture and further because the amount of moisture capable of being frozen, in different types of soil, differs widely.

The shrinkage of soils is an important characteristic, one directly responsible for serious cracking in concrete roads. It is frequently the case that evaporation of water from the sub-grade takes place more rapidly near the shoulders than at the center. Naturally, there is then more vertical movement due to soil shrinkage at the sides of the road than at the center, and, in some cases, as in the adobe soils of California, the sub-grade shrinks entirely away from the road slab so that a serious longitudinal crack develops. Such illustrations will suffice to demonstrate the important part which the sub-grade material must play in the successful design of a road surfacing.

*Physical Tests of Sub-Grade Materials.*—Most road engineers can recognize immediately the very worst types of soils and also the very best types; it is not always possible to distinguish good from bad soils when they lie somewhere in an intermediate range. One important investigation, then, has consisted in the development of tests that will distinguish the various soil types.

During the past several years, the United States Bureau of Public Roads has been working on the standardization of tests to distinguish and define the various properties of sub-grade materials. These tests may be listed as:

1.—Mechanical analysis.
2.—Water-holding capacity.
3.—Moisture equivalent.
4.—Volumetric change.
5.—Vertical capillarity.
6.—Comparative bearing value.
7.—Slaking value.
8.—Dye adsorption.

Mechanical Analysis.—This test consists of separating the sample by sieving and washing, into sand, silt, clay, and suspension-clay. Sand is that

part of the sample which passes a 10-mesh sieve and is retained on a 200-mesh sieve, but which settles out of a mixture of soil and water, after 8 min. subsidence, to a depth of 8 cm. from the surface of the liquid.

Silt is that part of a soil sample which passes a 200-mesh sieve and settles out of a mixture of soil and water, after 8 min. subsidence, to a depth of 8 cm. from the surface of the liquid.

Clay is that part of the soil sample which remains in suspension after 8 min. subsidence, to a depth of 8 cm. from the surface of the liquid, but which is thrown down when a centrifugal force equal to 500 times the force of gravity is exerted on the suspended material for a period of ½ hour.

Suspension-clay is that part of the soil sample which remains in suspension when a centrifugal force equal to approximately 500 times the force of gravity is exerted on the suspended material for a period of ½ hour. This is the very finest ingredient of the soil sample.

Water-Holding Capacity.—The water-holding capacity of the soil is the maximum percentage of water which it is capable of retaining.

Moisture Equivalent.—The moisture equivalent of the soil is the percentage of moisture which is retained by the soil after it has first been saturated and then subjected to a centrifugal force equal to 1 000 times the force of gravity for a period of 1 hour. This seems to be a very important test and is a measure of the difficulty of drainage.

Vertical Capillarity.—In this test, the percentage of moisture taken up in a given quantity of the soil by vertical capillarity is determined.

Comparative Bearing Value Test.—In this test, the load-supporting value of the soil is determined.

Volumetric Change.—This test is one to determine the amount of shrinkage that takes place as the soil changes from a condition of definite moisture to an air-dried condition.

Slaking Value.—In this test is determined the length of time required for a briquette of definite size to fall to pieces when immersed in water.

Dye Adsorption Test.—Different soils have the ability in greater or lesser degrees of decolorizing dye solutions which are filtered through them. The variability of dye adsorption properties by soils seems to be correlated with variability of other properties.

A complete description of the details of these tests has been published elsewhere.*

*Discussion of Physical Tests on Soils.*—It cannot yet be said that the results of the physical tests have been sufficiently correlated with the field behavior of sub-grade materials to establish the limits which define good or poor sub-grade materials.

Mechanical Analysis.—In general, it is quite evident from field observation that soils having a high percentage of the coarse particles are those which give the best results. Such soils, in general, have very low moisture capacity, low capillary absorption, low volumetric change, due to moisture changes, and comparatively high bearing value, no matter what their moisture content may

* "Physical Properties of Sub-Grade Materials", by J. R. Boyd, *Proceedings*, Am. Soc. for Testing Materials, Vol. 22 (1922).

be. On the other hand, soils having high percentages of the very finely divided materials, such as suspension-clay and clay, are the so-called "heavy" soils, which, in general, have high volumetric change, high capacity for moisture, and a high degree of plasticity when they become moist, even with capillary moisture. It remains to be established, however, what the safe limits of suspension-clay and clay might be in a soil for use under the road surface.

Water-Holding Capacity.—The water-holding capacity of a soil is related to the mechanical analysis, since soils with a high percentage of very fine particles, usually have high capacity for holding water. High water-holding capacity in a soil, therefore, is indicative of a poor sub-grade material for such soils are hard to drain and, moreover, generally, but not always, swell and shrink a great deal with changes in moisture. Because of their high moisture content, frost action is apt to cause considerable heaving of the overlying road surface.

Moisture Equivalent.—The moisture equivalent seems to be closely related to the mechanical grading and is apparently a measure of the difficulty of drainage. At the same time, it seems to be a test which gives promise of becoming almost a single measure of the sub-grade-making properties of a soil. Soils having high moisture equivalent are difficult to drain, have high moisture-holding capacity, are apt to be plastic when wet, to swell and shrink a great deal, and will probably heave unduly under frost action. At present, it is considered that moisture equivalent is perhaps the most significant test for sub-grade materials of all those described. There are some field investigations in the West which have indicated that a moisture equivalent of twenty may be taken, at least tentatively, as a rough dividing line between desirable and undesirable sub-grades.

Volumetric Change.—This is an important test for soils as it gives a direct indication of the vertical motion (and, unfortunately, non-uniform vertical motion), which is apt to accompany changes in moisture content of the sub-grade.

Vertical Capillarity.—Vertical capillarity is significant in connection with the height of the water-table under the road surface. If a soil has high vertical capillarity, it will absorb a considerable quantity of water from a free source of supply. High vertical capillarity in a sub-grade material is undesirable.

Comparative Bearing Value.—As it is a function of the sub-grade to support the road surfacing with comparatively little deformation, it is necessary to have relatively high bearing value in a sub-grade material. This question is one of considerable complexity and will be discussed later.

Slaking Value.—The slaking value of soils seems to be a measure of the speed with which the various materials will soften under the influence of water. It is not known yet, however, just how to interpret laboratory results in terms of field behavior.

Dye Adsorption.—Soils having a high percentage of colloidal material are capable of decolorizing dye solutions; the dye adsorption test, therefore, furnishes another means for determining to some degree the relative plasticity of soils. In general, soils showing high dye adsorption will have a high percentage

of volume change and, in general, are likely to be undesirable sub-grade materials. Tests show that the dye adsorption of the clay and suspension-clay sections of all soils is quite high, but differs considerably among the various parts. The clay and suspension-clay make for lack of stability.

It is quite probable that, in examining a sub-grade soil in the laboratory, all the previously described tests need not be used. It may be found that a sufficiently complete analysis results from the use of a few of these tests. Thus far, the moisture-equivalent test seems to have outstanding value, although it is probable that this test alone is not entirely sufficient. It seems to offer, however, a most direct means for identifying, tentatively at least, poor and good sub-grade soils. Further, it may be performed in the field in such manner as to give almost identical results with the more exact laboratory procedure.

*Field Method for Determining Moisture Equivalent of Soils.*—This test seems to have been originated by Mr. A. C. Rose, of the Bureau of Public Roads. The procedure consists of drying a 500-gramme sample of soil at a temperature lower than 100° cent., breaking up the lumps, placing the sample in a bowl, and slowly introducing water from a burette, mixing the water and soil thoroughly together to the consistency of putty. Water is then allowed to drop on the smoothed off surface of the sample and spread with a spoon or spatula. It will be found that before the moisture-equivalent value is reached, the wet, shiny appearance will disappear from the soil surface almost immediately after the drop of water is smoothed over the surface; after the critical value is reached, the surface will retain its wet, shiny appearance. The sample is then dried out and the percentage of water is calculated on the basis of the dry weight. After 0.8% of water is added to this value, the result will be the moisture equivalent of the soil. Tentatively, it can be stated that the approximate dividing line between bad sub-grade soils and good sub-grade soils lies in the moisture-equivalent range of 15 to 20.

*Influence of the Bearing Value of the Sub-Grade.*—The intensity of pressure on a road sub-grade is governed by a number of factors. The wheel load is carried directly by the wearing surface which, in turn, transmits the pressure to the underlying sub-grade. If the wearing surface is of a rigid type, as concrete, capable of resisting bending, the sub-grade pressures are distributed over a comparatively broad area adjacent to the wheel, and the highest intensity of pressure is comparatively low. On the other hand, if the surfacing material is of a flexible type, incapable of resisting bending, the area of distribution of sub-grade pressures will be quite restricted and the intensity of pressure under the wheel will be high. These statements are illustrated by curves of pressure distribution obtained under concrete and with broken-stone surfaces of equal thickness, both subjected to a concentrated load. (Fig. 1.) The pressures were measured by soil pressure cells, described elsewhere.*

The degree of yielding of the soil also affects the pressure distribution. Thus, a solid rock sub-grade would have high and restricted pressures under the wheel load, whereas a soft soil would have low pressure intensities, well spread out, provided, of course, the over-lying slab was of such a nature as

* "The Distribution of Pressure Through Earth Fills", by A. T. Goldbeck, *Proceedings*, Am. Soc. for Testing Materials, 1917.

to resist bending. The flexibility of the slab likewise controls the pressure intensity and distribution. Rigid slabs, such as concrete, can deflect little before a crack is produced; other types of pavements of less rigid character can be deformed more, without the accompanying incipient failure. The sub-grade material, therefore, has to develop its resistance to the over-lying load within safe limits of deformation of the pavement. It is quite important that the bearing value be obtained corresponding to a known amount of deformation of the sub-grade. This amount will be less with rigid slabs than with non-rigid slabs.

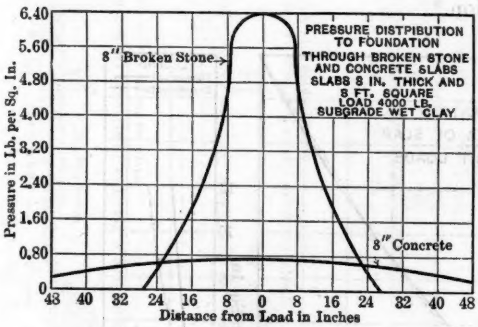



FIG. 1.

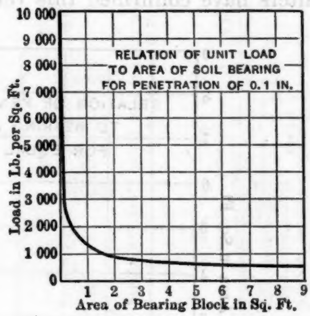



FIG. 2.

*Influence of Bearing Area on Bearing Value of Soils.*—Another consideration in determining the bearing value of sub-grades is that due to the different areas of support offered by the sub-grade, depending on the rigidity of the road slab. The bearing value of a soil, for a given penetration of the bearing area, depends on the magnitude of that area. It was with these considerations in mind that a series of tests were conducted to determine the influence of the size of areas on the bearing value of soils for definite penetrations of those bearing areas into the soil. These investigations were conducted using bearing blocks of from several square inches up to 9 sq. ft. in area, and with different types of sub-grade materials, such as plastic clay and sand. Although there is some variation in the law for the different kinds of materials, the accompanying curve, Fig. 2, is, in general, indicative of results obtained. It will be noted that when small bearing areas are used, the intensity of pressure required to produce a penetration of 0.1 in. far exceeds that for large sized blocks. This is readily explained by the fact that when the large bearing areas are used a greater thickness of the soil is compressed, which contributes more toward the movement of the block than in the case of the small bearing area. It may not always be possible to make bearing value determinations by the use of large size tests. If smaller bearing blocks are used, it will be necessary to obtain the bearing value of the soil at a much less penetration, in order to obtain values corresponding to a definite penetration of the large bearing block. The relation between the depths of penetration of different sized bearing blocks for like intensities of pressure has been established by a series of tests using various bearing areas.* In Fig. 3 is shown the results of this investigation, giving

* The details of this investigation have not yet been published.

the relative penetrations of the areas under equal intensities of load. The experimental results for a penetration of 0.1 in. over the area of 9 sq. ft. seem to be quite accurately expressed by the relation:

$$\text{Area (square feet)} = 900 \times [\text{penetration (inches)}]^2$$

From this equation, it follows that:

$$\frac{\text{Penetration }(a)}{\text{Penetration }(A)} = \sqrt{\frac{a}{A}}$$

in which, $a$ and $A$ are the areas of the respective bearing blocks. Other experimenters have confirmed this relation.*

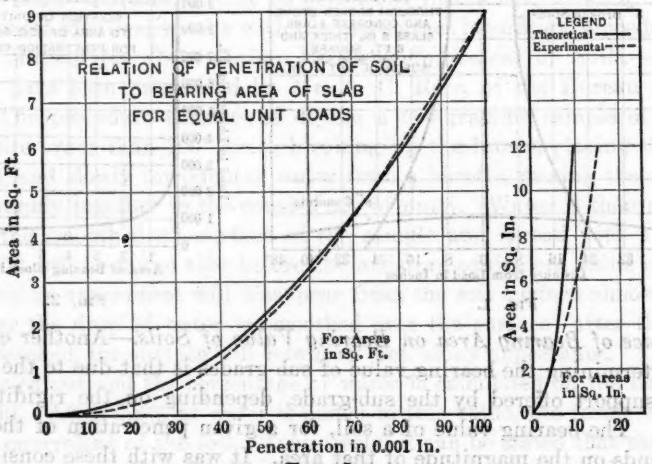



FIG. 3.

*Experiments in Sub-Grade Moisture.*—Even the worst kind of sub-grade material has a high bearing value when it is dry. On the other hand, the best kind of sub-grade materials are in their best condition when slightly moist; this applies especially to clean sand. The worst sub-grade materials are dangerous only when they have taken up a sufficient quantity of moisture to cause them to swell or to render them plastic. It is important, therefore, that complete information be obtained as to the laws governing moisture in soils.

*Drainage Experiments.*—In an effort to determine the effectiveness of a number of different systems of drainage, slabs, 14 ft. sq., were laid as shown in the cross-sections of Fig. 4. It will be noted that the sub-grades under these slabs were tile-drained in some cases, that all had deep side ditches, and that several had deep impervious walls surrounding the slabs. Each slab was perforated with holes through which samples of the underlying sub-grade could be obtained. The accompanying data (Table 1) gives the results of these moisture determinations for each of the various sections.

As a result of these experiments, it is concluded that systems of drainage merely remove the free water in the sub-grade, but cannot remove capillary moisture, and it seems that notwithstanding the different systems of drainage

* See article in *Le Genie Civil,* May 26, 1923, by Bijls, of Belgium.

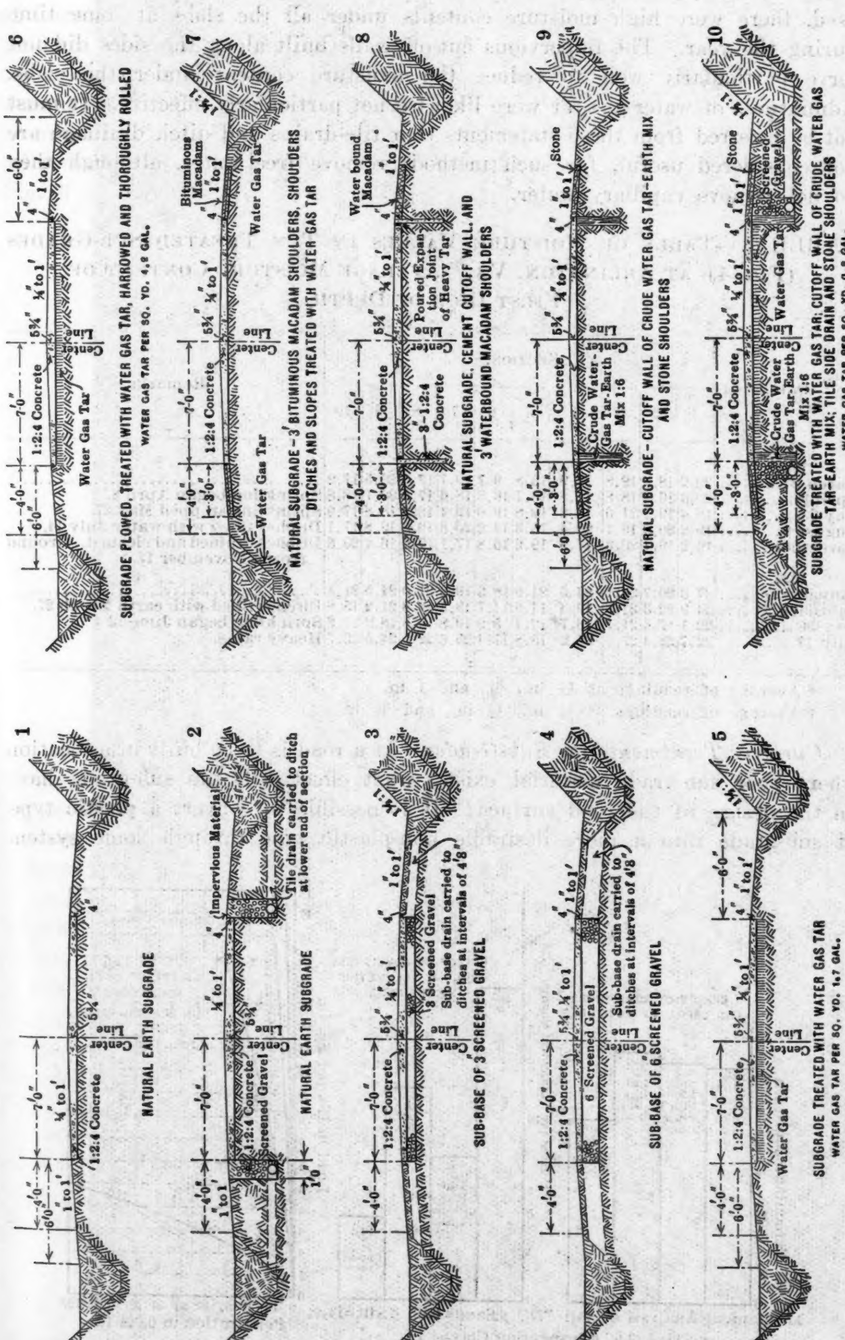



FIG. 4.—CROSS-SECTIONS, PAVEMENT TYPES 1 TO 10, FOR DRAINAGE AND SUB-GRADE EXPERIMENTS.

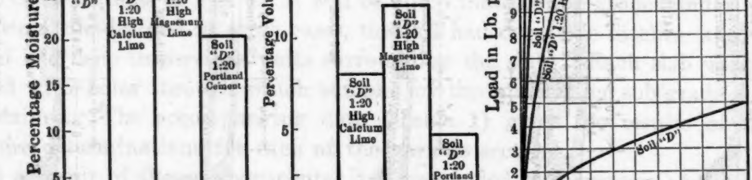
used, there were high moisture contents under all the slabs at some time during the year. The impervious cut-off walls built along the sides did not serve particularly well to reduce the moisture content under the slabs. Admixtures of water-gas tar were likewise not particularly effective. It must not be inferred from these statements that tile-drains and ditch drainage are not considered useful, for such methods remove free water, although they do not remove capillary water.

TABLE 1.—TABLE OF MOISTURE VALUES IN TEN TREATED SUB-GRADES (FIG. 4), AT ARLINGTON, VA. AVERAGE MOISTURE CONTENT OF FIRST INCH OF DEPTH.

| Date. | Section. | | | | | | | | | | Remarks. |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | 10 | |
| 1921. | | | | | | | | | | | |
| March 2 | 20.2 | 18.9 | 19.8 | 20.9 | 16.8 | 9.7 | 19.7 | 17.6 | 20.5 | 17.2 | ........ |
| April 18 | 20.4 | 16.2 | 18.8 | 16.8 | 16.7 | 16.3 | 18.6 | 17.6 | 20.1 | 18.8 | Sprinkling began April 8. |
| May 6 | 18.4 | 19.5 | 21.0 | 19.2 | 19.8 | 19.6 | 19.4 | 18.3 | 18.8 | 17.9 | Sprinkling stopped May 27. |
| August 5 | 19.2 | 20.0 | 19.1 | 20.4 | 15.3 | 14.2 | 20.8 | 19.8 | 19.8 | 17.1 | Ditches filled with water July 14. |
| November 7 | 16.2 | 19.1 | 20.3 | 17.9 | 19.2 | 16.8 | 17.1 | 17.2 | 16.4 | 20.6 | Ditches drained and cleaned. Ground frozen November 17. |
| 1922. | | | | | | | | | | | |
| January 25 | 37.3 | 20.7 | 24.1 | 24.6 | 21.6 | 18.2 | 19.2 | 21.0 | 24.5 | 21.4 | ........ |
| February 20 | 24.9 | 22.3 | 21.8 | 19.3 | 14.6 | 14.7 | 18.5 | 20.8 | 21.2 | 15.8 | Ditches filled with earth March 27. |
| May 23 | 22.1 | 17.6 | 21.1 | 18.7* | 12.4 | 5.9 | 19.8 | 16.8 | 18.2 | 17.2 | Sprinkling began June 12. |
| July 17 | 22.7 | 23.4 | 27.0 | 25.2† | 15.8 | 17.4 | 25.6 | 22.5 | 24.5 | 22.7 | Heavy rains. |

* Average of readings at ¼ in., ¾, and 1 in.
† Average of readings at ¼ in., ½ in., and ¾ in.

*Curative Treatments for Sub-Grades.*—If a road is to be built in a location where poor sub-grade material exists, what effect will the sub-grade have on the design of the road surface? Is it possible to convert a plastic type of sub-grade into a more desirable non-plastic type through some system




Mechanical Analysis of Soil "D" % Sand=40.8; % Silt=14.9; % Clay=39.9; % Suspension Clay=4.4

FIG. 5.

of curative treatment? Experiments to determine these points have been conducted using various admixtures to soil, with the results shown in Fig. 5, from which it appears that 5% additions of hydrated lime and Portland cement have the effect of decreasing the volumetric change due to variations in moisture content and likewise of increasing the bearing value of plastic soils that contain moisture up to a limit of their capillary moisture. In Fig. 6 is shown the effect of additions of different quantities of sand for

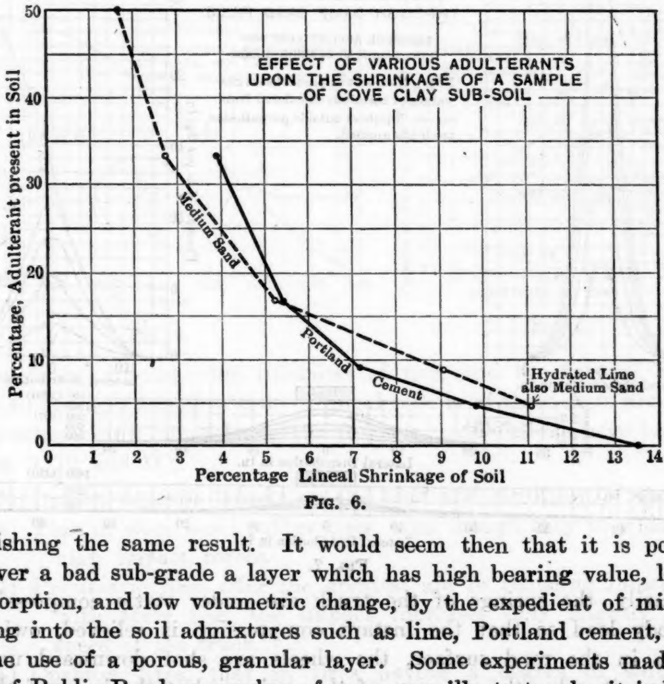



FIG. 6.

accomplishing the same result. It would seem then that it is possible to create over a bad sub-grade a layer which has high bearing value, low moisture adsorption, and low volumetric change, by the expedient of mixing and harrowing into the soil admixtures such as lime, Portland cement, or sand, or by the use of a porous, granular layer. Some experiments made by the Bureau of Public Roads a number of years ago illustrate why it is beneficial to use a layer of material of high supporting value interposed between the plastic sub-grade and the overlying road slab. The resulting curves (Fig. 7) show the distribution of pressure intensity due to concentrated loads placed on top of layers of different thicknesses. As would be expected, the thicker the layer, the smaller is the intensity of pressure at the bottom and the wider is that pressure spread. Hence, a layer of material of good supporting value has the effect of decreasing the intensity of pressure on the soft sub-grade due to concentrated loads on the road surface. Under just what conditions it is economical to use such a layer, created either by admixing with the sub-grade, or by placing on top of the sub-grade, is yet to be demonstrated.

## LOADS ON PAVEMENTS

It is quite necessary to know not only the static weights of the motor vehicles operating over the highways, but also the impact pressures produced on pavements by trucks moving at high speeds and equipped with a wide

variety of tires. In a large number of tests of this nature, an effort has been made to obtain the relative impacts under the various conditions that might exist due to road roughness. In Fig. 8 is illustrated the motion both of the body of the truck and of the unsprung portion when the rear wheels fall from a definite height and strike the road surface.

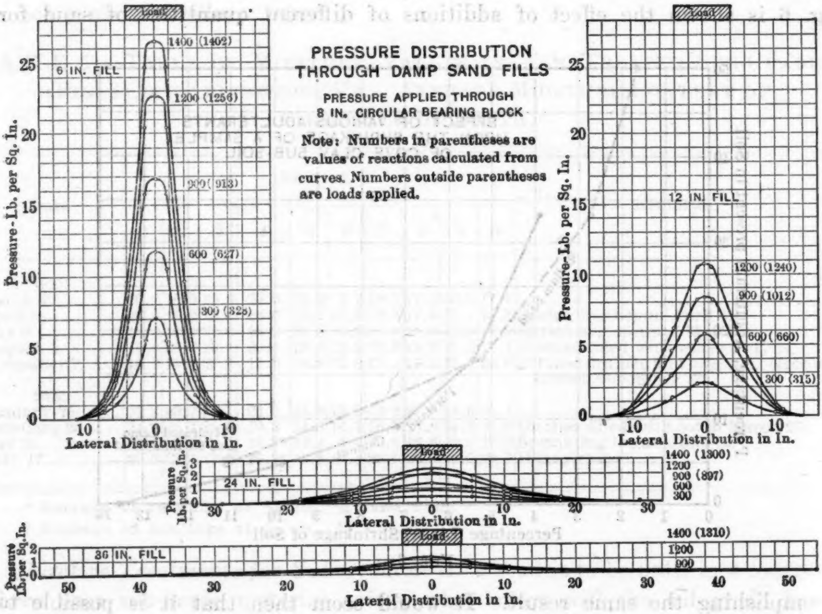


FIG. 7.

Ordinarily, the springs of the truck are under static compression, due to the body load so that the instant compression is relieved, owing to a depression in the road surface, the wheels are shot downward under the force of gravity plus the pressure of the springs; but the body of the truck descends much more slowly. At the instant the tires come into contact with the road surface again, upward pressure on the wheels begins and downward vertical motion of the wheels is finally reduced to zero. The pressure on the road surface has increased from zero at the instant of initial contact with that surface, to a maximum when the tire is fully compressed. To obtain a measure of this pressure, the following method of test was used.

Cylinders were prepared in large numbers by cutting ½-in. lengths from copper bars ½ in. in diameter. All the cylinders were thoroughly annealed under the same conditions, in order to render them equally resistant to loads. A hydraulic jack was set on a concrete base and the plunger, provided with a platform, was set at about the same level as the road surface, one of the annealed copper cylinders being then placed under the plunger of the jack. The motor truck was driven so that its rear wheels came into contact with this plunger, and the full force of the blow was delivered to the copper cylinder, thus permanently deforming it. The pressures corresponding to

various cylinder shortenings having been determined separately, the effect of the blow was readily known in terms of these deformations. In some cases, the truck wheels mounted a slight incline and dropped from definite heights to the plunger; in other cases, obstructions were placed on the plunger. A large variety of truck tires were used on the same truck, with like conditions of loading, and a number of different kinds of trucks, operated over a wide range of speed. In this manner, the influence of the weight of the truck, the sprung and the unsprung weight, the speed, kind of tire—whether pneu-

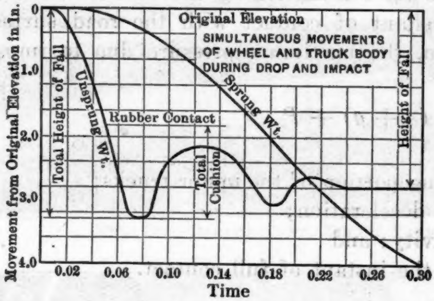



FIG. 8.

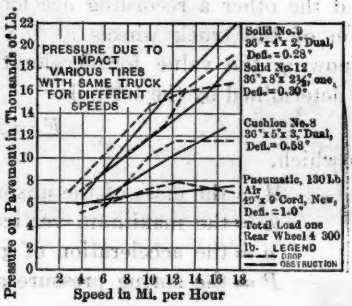



FIG. 9.

matic, cushion, or solid,—the thickness of tire, and the type of inequality in the road surface, was obtained. Typical results of these tests are indicated in the accompanying curves* and the following conclusions have been drawn (see Figs. 9, 10, and 11):

(1) Impact depends largely on the kind and condition of the tire.

(2) Thin or worn solid rubber tires, even though they be wide, produce high impact forces.

(3) Pneumatic tires offer the greatest aid in reducing impact forces; with their use, the impact increases only slightly with the speed of the truck.

(4) Cushion tires, that is, tires having a degree of softness and deflection between solid and pneumatic types, offer corresponding advantages in reducing impact.

(5) Impact increases with the speed of the truck, but it cannot be said to increase according to any constant ratio or power of the speed.

(6) Although heavy unsprung weight may give higher impact than lighter unsprung weight, it cannot be said that this is the major controlling factor.

(7) The relative destructive effect produced by light weight, high-speed trucks, and heavy, slow-moving trucks has not been determined by these tests. They do indicate, however, that equal impact may be obtained under some conditions.

(8) Impact may be as high as seven times the static load on one rear wheel when a solid-tired truck strikes a 1-in. obstruction at 16

* See, also, *Public Roads,* March and December, 1921.

miles per hour, an average value being about four times. For pneumatic tires, the maximum impact value is probably not more than 1¾ times the load on one rear wheel, and an average value is not more than 1¼ times the load.

(9) All cushion wheels do not reduce impact on the road surface even though they may cushion the vehicle.

Other devices are now being used in similar investigations with a promise of obtaining successful results. One of these devices is the Kreüger cell* and the other a recording accelerometer for measuring the vertical deceleration of the truck wheels at the instant of contact with the road surface. Knowing the value for deceleration, the maximum pressure due to impact is determined by the formula:

$$F = M\,(a + g) + P$$

in which,

$M$ = the mass of the unsprung portion of the motor trucks;
$a$ = the maximum vertical deceleration;
$g$ = the acceleration of gravity; and
$P$ = the spring pressure at the instant of full contact.

## Stresses in Concrete Pavements

Another series of tests leading up to the rational design of concrete roads has been that of the determination of the distribution of maximum fiber deformation in the surface of the road slab due to motor-truck traffic. For this purpose, it was necessary to design a special strain-gauge which would record these maximum deformations. It was necessary that this instrument be of the recording type, small and compact.

The type finally evolved (Fig. 12), called a "graphic strain-gauge", consists of a single lever having a multiplication of approximately 70 to 1, the long arm carrying a needle which scratches a mark on a smoked-glass plate. This mark is approximately seventy times as long as the movement of the plunger of the instrument, which movement equals the fiber deformation. The glass record is "fixed" in a thin varnish solution and then examined under a microscope to obtain the length of the scratch mark. Very consistent results have been obtained with this instrument under static and dynamic loads, and it is known to have practically no "overthrow" under dynamic loads. When used for stress measurements, a slot is cut in the surface of the pavement, ¾ in. square and about 6½ in. long, just large enough to receive the instrument the ends of which bear against brass plates cemented at the ends of the slot. Finally, a thin steel plate is laid on the pavement over the instrument to protect it against possible disturbance from the motor-truck tires. A number of gauges being thus placed at various locations in the surface, the motor truck is run over the pavement a number of times in different positions, and after each run the smoked-glass plate in each instrument is moved forward a slight amount, placing it in position to

* "Method for Measuring and Calculating the Magnitude of Forces with Particular Regard to Impact Forces," by H. Kreüger, *Transactions No. 2*, Eng. Science Academy, Stockholm, Sweden.

FIG. 10.—IMPACT TEST, TRUCK PASSING OVER MEASURING APPARATUS.

FIG. 11.—DETAILS OF COPPER CYLINDER DEVICE.

FIG. 12.—GRAPHIC STRAIN GAUGE.

receive another record. In this manner, a dozen or more records may be inscribed on the same smoked glass.

In considering slab design, the maximum deformations in the concrete are of greatest interest, irrespective of the position of the load which causes those maxima. For the purposes of the present determination, the outer wheels of the trucks were run at different distances from the sides of the road. In a plain concrete slab of 6-in. uniform cross-section, the maximum fiber deformations using a 5-ton truck were as shown in Fig. 13.

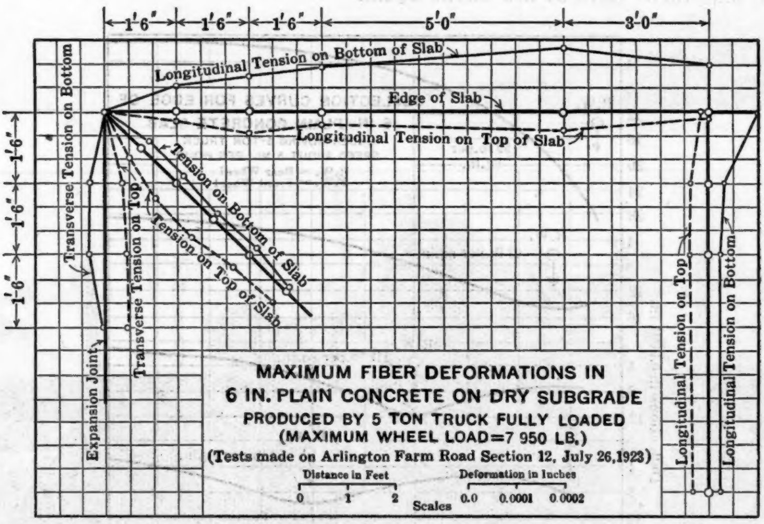



FIG. 13.

It will be noted that there is considerable tension in the bottom of the slab at the edges and also that there is considerable tension on a 45° diagonal in the top of the slab at a corner formed by the intersection of a transverse joint and the edge. The longitudinal stress in the interior of the slab directly under the wheel load is much less than that at the outer edge, and it would seem certain, therefore, that strengthening of the edge of the slab is necessary to obtain a slab of more nearly balanced design. It is to be noted that the tension in the bottom of the slab along the edge approximates the tension in the top near a corner.

*Deflection Curves Along the Side of a Concrete Road.*—An autographic deflectometer, built specially for the purpose, has been used for obtaining a measure of the deflections along the side of road slabs while taking stress measurements as just described. A view of this instrument is shown in Fig. 15. It consists of a board, 20 ft. long, carrying a series of pen arms actuated by plungers resting on the pavement. The travel of the pen is 100 times the deflection of the pavement and the record is made on a strip of paper moved by an electric motor. As the trucks run past the instrument, each pen arm traces a curve giving an exaggerated graphic representation of the deflection; from the twenty curves thus drawn, the true shape of the

elastic curve is obtained. A series of deflection curves, Fig. 14, is shown for a 6-in. slab; similar curves have been obtained for slabs of several different designs. They corroborate the strain-gauge indications that high tension may exist both at the top and bottom of the slab along the edge.

*Stress Along Center Joint of a Concrete Road.*—The question has often been raised as to whether the stresses due to wheel loads are likely to be as high along an unsupported center longitudinal joint in a concrete road as along the edge of the slab. Measurements made on a 6-in. concrete road show that these stresses are about equal.

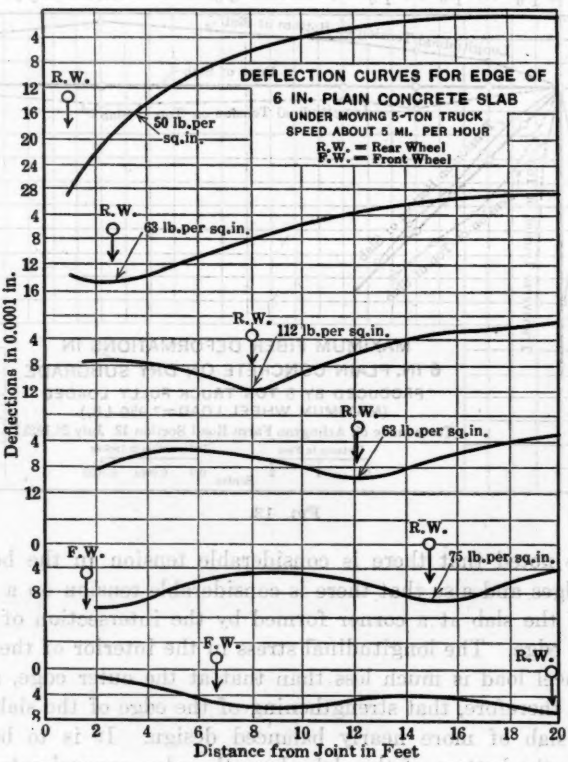



FIG. 14.

*Effectiveness of Dowels in Transmitting Load Across a Center Joint.*—If the stress created along a center joint by a wheel load is high, the question arises as to whether dowels may be effectively used to transmit it across to the adjacent slab and thus bring both slabs into action. On viewing the deflection curves of Fig. 14, it is seen that the sudden curvature takes place over a few feet. High stress exists where there is sharp curvature in the elastic line; therefore, dowels, to be effective, must be placed at sufficiently close intervals to eliminate this sharp curvature.

*Impact Tests on Pavement Slabs.*—In order to obtain a relative measure of the resistance of the many types and designs of pavements to the action

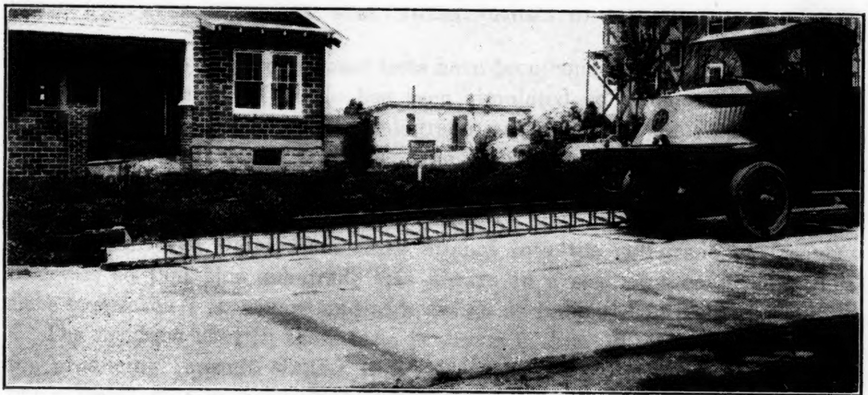


FIG. 15.—VIEW OF INSTRUMENT FOR RECORDING PAVEMENT DEFLECTIONS.

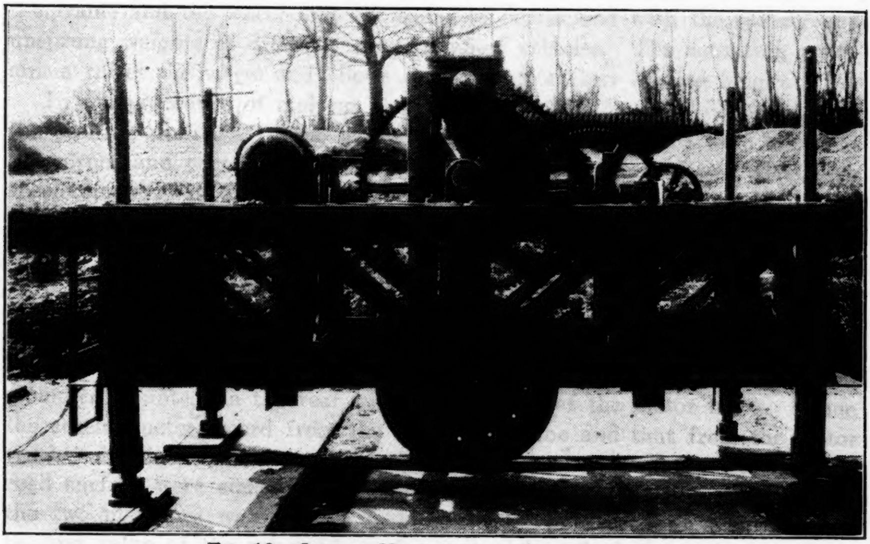


FIG. 16.—IMPACT MACHINE FOR TESTING SLABS.

FIG. 17.—WEAR MACHINE USED AS CONCRETE TRUCK.

Fig. 15.—Mobile Instrument for Recording Pavement Deflections.

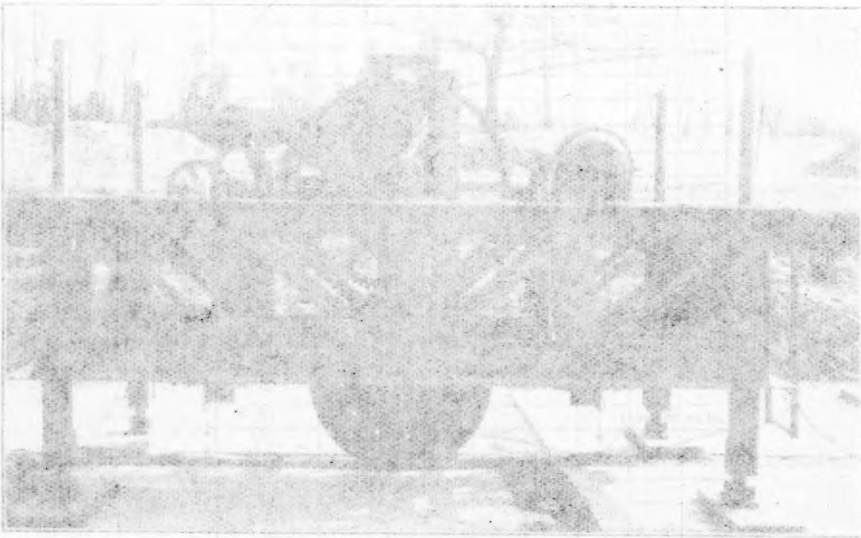


Fig. 16.—Impact Machine for Testing Slabs.

Fig. 17.—Wear Machine Used as Concrete Track.

of traffic, a large series of impact tests have been conducted at Arlington, Va., in which the action of traffic has been simulated as nearly as practicable.

Before laying the slabs, the sub-grade was carefully prepared so that it would be as uniform as possible. One series of specimens was laid on a sub-grade having side ditches which were kept dry, and a duplicate series with side ditches which were kept constantly filled with water. Tile-drains likewise conveyed water from these ditches into the sub-grade at frequent intervals so that this sub-grade was always in a soft, wet condition. The slabs were each 7 ft. square and of a design as indicated in Table 2.

The machine used in these tests is shown in Fig. 16. It is a mechanism for producing impacts similar to those of actual motor trucks and has a truck wheel that may be shod with any kind of tire desired, and a truck spring interposed between the frame of the machine and the wheel. Both the frame and the wheel may be loaded to correspond with the sprung and unsprung weights of different sizes of motor vehicles. The actuating mechanism raises the wheel and allows it to fall from any desired height.

In the procedure of making these tests first, the machine is mounted in the position desired. In the present series, part of the slabs were tested at the corner and part with the wheel placed at the middle of one edge. An accelerometer was mounted on the unsprung portion of the machine, and the specimens were fitted with graphic strain-gauges of the type already described, also with Ames dials for deflection and autographic means for obtaining vertical movements of both the slab and the wheel. At first, impacts were delivered to simulate a 2-ton truck, beginning with low heights of fall and increasing to a height which produced impacts as high as had been discovered by tests of actual traffic, as determined by the readings of an accelerometer mounted on the rear unsprung portion of the motor truck. When the accelerometer record from the impact machine and that from the motor truck coincided, it was assumed that the forces received by the slab and road surface were equal, inasmuch as the sprung and unsprung weights of the two machines were alike. Auxiliary tests were used to determine the bearing value of the sub-grade under each slab and also the physical characteristics of the concrete. This investigation will not be reported in detail; the results, however, are given in Figs. 18, 19, 20, 21, 22, and 23.

It is not intended that these tests shall be used to indicate the actual resistance of the various designs of pavements to traffic loads, but rather the relative value of these various types for carrying traffic under the sub-grade conditions at Arlington. It is not unlikely that, in some cases, these relative resistances might vary considerably as the relation of support offered by the sub-grade and that offered by the slab changes. The tests, however, seem to warrant the following conclusions:

1.—The resistance of a road slab depends in part on the supporting value of the sub-grade. A sub-grade of high supporting value materially increases the resistance to impact.

2.—The resistance of rigid slabs to impact varies neither directly with the depth nor as the square of the depth, but as some power less than two.

## TABLE 2.—Slabs for Impact Tests.

**(All Slabs are 7 Ft. Square of Material and Thickness, as Shown Below. Series 200-267, on Wet Sub-Grade; Series 300-337, on Dry Sub-Grade; Series 1R-18R, on Dry Sub-Grade.)**

Plain Slabs.

| Slab no. | Base. | | Binder. | | Surface. | |
|---|---|---|---|---|---|---|
| | Thickness, in inches. | Material. | Thickness, in inches. | Material. | Thickness, in inches. | Material. |
| 200 | 6 | Macadam | 4 | Bituminous concrete | 2 | Topeka |
| 201 | 6 | " | 4 | " " | 2 | " |
| 204 | 4 | " | 3 | " " | 2 | " |
| 205 | 4 | " | 3 | " " | 2 | " |
| 208 | 4 | " | 2 | .......... | 2 | " |
| 209 | 4 | " | 2 | .......... | 2 | .......... |
| 212 | 6 | " | .... | .......... | 2 | Topeka |
| 213 | 6 | " | .... | .......... | 2 | " |
| 214 | 9 | " | .... | .......... | 2 | " |
| 215 | 9 | " | .... | .......... | 2 | " |
| 216 | 12 | " | .... | .......... | 2 | " |
| 217 | 12 | " | .... | .......... | 2 | " |
| 218 | 6 | Bituminous concrete | .... | .......... | 2 | " |
| 219 | 6 | " " | .... | .......... | 2 | " |
| 220 | 4 | " " | .... | .......... | 2 | " |
| 221 | 4 | " " | .... | .......... | 2 | " |
| 222 | 6 | 1 : 3 : 6 K* | 1½ | Bituminous concrete | 1½ | Asphalt top |
| 223 | 6 | 1 : 3 : 6 K | 1½ | " " | 1½ | " " |
| 224 | 6 | 1 : 3 : 6 K | .... | .......... | 4 | C. Bituminous concrete |
| 225 | 6 | 1 : 3 : 6 K | .... | .......... | 4 | " " " |
| 226 | 6 | 1 : 3 : 6 K | .... | .......... | 2 | " " " |
| 227 | 6 | 1 : 3 : 6 K | .... | .......... | 2 | " " " |
| 228 | 4 | 1 : 3 : 6 K | .... | .......... | 2 | Topeka |
| 229 | 4 | 1 : 3 : 6 K | .... | .......... | 2 | " |
| 230 | 4 | 1 : 1½ : 3 K | .... | .......... | 2 | " |
| 231 | 4 | 1 : 1½ : 3 K | .... | .......... | 2 | " |
| 232 | 6 | 1 : 3 : 6 K | .... | .......... | 2 | " |
| 233 | 6 | 1 : 3 : 6 K | .... | .......... | 2 | " |
| 234 | 6 | 1 : 1½ : 3 K | .... | .......... | 2 | " |
| 235 | 6 | 1 : 1½ : 3 K | .... | .......... | 2 | " |
| 236 | 8 | 1 : 3 : 6 K | .... | .......... | 2 | " |
| 237 | 8 | 1 : 3 : 6 K | .... | .......... | 2 | " |
| 238 | 8 | 1 : 1½ : 3 K | .... | .......... | 2 | " |
| 239 | 8 | 1 : 1½ : 3 K | .... | .......... | 2 | " |
| 240 | .. | .......... | .... | .......... | 4 | 1 : 1½ : 3 K |
| 241 | .. | .......... | .... | .......... | 4 | 1 : 1½ : 3 K |
| 242 | .. | .......... | .... | .......... | 6 | 1 : 1½ : 3 K |
| 243 | .. | .......... | .... | .......... | 6 | 1 : 1½ : 3 K |
| 244 | .. | .......... | .... | .......... | 6 | 1 : 3 : 6 K |
| 245 | .. | .......... | .... | .......... | 6 | 1 : 3 : 6 K |
| 246 | .. | .......... | .... | .......... | 8 | 1 : 1½ : 3 K |
| 247 | .. | .......... | .... | .......... | 8 | 1 : 1½ : 3 K |
| 248 | .. | .......... | .... | .......... | 8 | 1 : 3 : 6 K |
| 249 | .. | .......... | .... | .......... | 8 | 1 : 3 : 6 K |
| 300 | 4 | Macadam | 2 | Bituminous concrete | 2 | Topeka |
| 301 | 4 | " | 2 | " " | 2 | " |
| 304 | 4 | " | .... | .......... | 2 | " |
| 305 | 4 | " | .... | .......... | 2 | " |
| 306 | 6 | " | .... | .......... | 2 | " |
| 307 | 6 | " | .... | .......... | 2 | " |
| 308 | 9 | " | .... | .......... | 2 | " |
| 309 | 9 | " | .... | .......... | 2 | " |
| 310 | 6 | Bituminous concrete | .... | .......... | 2 | " |
| 311 | 6 | " " | .... | .......... | 2 | " |
| 312 | 4 | " " | .... | .......... | 2 | " |
| 313 | 4 | " " | .... | .......... | 2 | " |
| 314 | 6 | 1 : 1½ : 3 K | .... | .......... | 2 | " |
| 315 | 6 | 1 : 1½ : 3 K | .... | .......... | 2 | " |
| 316 | 6 | 1 : 3 : 6 K | .... | .......... | 2 | " |

*K = Cement concrete.

## TABLE 2.—(*Continued*).

PLAIN SLABS (*Continued*).

| Slab no. | Base. | | Binder. | | Surface. | |
|---|---|---|---|---|---|---|
| | Thickness, in inches. | Material. | Thickness, in inches. | Material. | Thickness, in inches. | Material. |
| 317 | 6 | 1 : 3 : 6 K* | .... | ......... | 2 | Topeka |
| 318 | 4 | 1 : 1½ : 3 K | .... | ......... | 2 | " |
| 319 | 4 | 1 : 1½ : 3 K | .... | ......... | 2 | " |
| 320 | 8 | 1 : 1½ : 3 K | .... | ......... | 2 | " |
| 321 | 8 | 1 : 1½ : 3 K | .... | ......... | 2 | " |
| 322 | .. | ........... | .... | ......... | 4 | 1 : 1½ : 3 K |
| 323 | .. | ........... | .... | ......... | 4 | 1 : 1½ : 3 K |
| 324 | .. | ........... | .... | ......... | 6 | 1 : 1½ : 3 K |
| 325 | .. | ........... | .... | ......... | 6 | 1 : 1½ : 3 K |
| 326 | .. | ........... | .... | ......... | 6 | 1 : 3 : 6 K |
| 327 | .. | ........... | .... | ......... | 6 | 1 : 3 : 6 K |
| 328 | .. | ........... | .... | ......... | 8 | 1 : 1½ : 3 K |
| 329 | .. | ........... | .... | ......... | 8 | 1 : 1½ : 3 K |

SLABS WITH MESH REINFORCING. WET SUB-GRADE.

| Slab no. | Surface. | | | |
|---|---|---|---|---|
| | Thickness, in inches. | Material. | Reinforcement. | Percentage of steel = $p$. |
| 250 | 4 | 1 : 1½ : 3 K | 1 layer No. 6 | 0.21 |
| 251 | 4 | 1 : 1½ : 3 K | 1 " " 6 | 0.21 |
| 252 | 4 | 1 : 1½ : 3 K | 2 " " 6 | 0.42 |
| 253 | 4 | 1 : 1½ : 3 K | 2 " " 6 | 0.42 |
| 254 | 4 | 1 : 1½ : 3 K | 2 " " 8 | 0.66 |
| 255 | 4 | 1 : 1½ : 3 K | 2 " " 8 | 0.66 |
| 256 | 4 | 1 : 1½ : 3 K | 1 " " 10 | 0.41 |
| 257 | 4 | 1 : 1½ : 3 K | 1 " " 10 | 0.41 |
| 258 | 4 | 1 : 1½ : 3 K | 2 " " 10 | 0.82 |
| 259 | 4 | 1 : 1½ : 3 K | 2 " " 10 | 0.82 |
| 260 | 6 | 1 : 1½ : 3 K | 1 " " 6 | 0.19 |
| 261 | 6 | 1 : 1½ : 3 K | 1 " " 6 | 0.19 |
| 262 | 6 | 1 : 1½ : 3 K | 2 " " 6 | 0.38 |
| 263 | 6 | 1 : 1½ : 3 K | 2 " " 6 | 0.38 |
| 264 | 6 | 1 : 1½ : 3 K | 2 " " 8 | 0.46 |
| 265 | 6 | 1 : 1½ : 3 K | 2 " " 8 | 0.46 |
| 266 | 6 | 1 : 1½ : 3 K | 2 " " 10 | 0.58 |
| 267 | 6 | 1 : 1½ : 3 K | 2 " " 10 | 0.58 |

SLABS WITH MESH REINFORCING. DRY SUB-GRADE.

| Slab no. | Surface. | | | |
|---|---|---|---|---|
| | Thickness, in inches. | Material. | Reinforcement. | Percentage of steel = $p$. |
| 330 | 4 | 1 : 1½ : 3 K | 1 layer No. 6 | 0.21 |
| 331 | 4 | 1 : 1½ : 3 K | 1 " " 6 | 0.21 |
| 332 | 4 | 1 : 1½ : 3 K | 1 " " 10 | 0.41 |
| 333 | 4 | 1 : 1½ : 3 K | 1 " " 10 | 0.41 |
| 334 | 6 | 1 : 1½ : 3 K | 2 " " 6 | 0.38 |
| 335 | 6 | 1 : 1½ : 3 K | 2 " " 6 | 0.38 |
| 336 | 6 | 1 : 1½ : 3 K | 2 " " 8 | 0.46 |
| 337 | 6 | 1 : 1½ : 3 K | 2 " " 8 | 0.46 |

* K = Cement concrete.

TABLE 2.—(*Continued*).

| Slab no. | Surface. | | | |
|---|---|---|---|---|
| | Thickness, in inches. | Material. | Reinforcement. | Percentage of steel = $p$. |
| 1 R | 6 | 1 : 1½ : 3 K* | 1 layer ⅜-in. rods at 3¾ in., cu. cm. | 0.5 |
| 2 R | 6 | 1 : 1½ : 3 K | 1 " " " " " " " " | 0.5 |
| 3 R | 6 | 1 : 1½ : 3 K | 2 " " " " " " " " | 1.0 |
| 4 R | 6 | 1 : 1½ : 3 K | 2 " " " " " " " " | 1.0 |
| 5 R | 6 | 1 : 1½ : 3 K | 2 " " " " " 7⅜ " " | 0.5 |
| 6 R | 6 | 1 : 1½ : 3 K | 2 " " " " " " " " | 0.5 |
| 7 R | 6 | 1 : 1½ : 3 K | 1 " ⅝ " " " 10¼ " " | 0.5 |
| 8 R | 6 | 1 : 1½ : 3 K | 1 " " " " " " " " | 0.5 |
| 9 R | 6 | 1 : 1½ : 3 K | 2 " " " " " " " " | 1.0 |
| 10 R | 6 | 1 : 1½ : 3 K | 2 " " " " " " " " | 1.0 |
| 11 R | 6 | 1 : 1½ : 3 K | 2 " " " " " 20½ " " | 0.5 |
| 12 R | 6 | 1 : 1½ : 3 K | 2 " " " " " " " " | 0.5 |
| 13 R | 6 | 1 : 1½ : 3 K | 1 " ⅞ " " " 20 " " | 0.5 |
| 14 R | 6 | 1 : 1½ : 3 K | 1 " " " " " " " " | 0.5 |
| 15 R | 6 | 1 : 1½ : 3 K | 2 " " " " " " " " | 1.0 |
| 16 R | 6 | 1 : 1½ : 3 K | 2 " " " " " " " " | 1.0 |
| 17 R | 6 | 1 : 1½ : 3 K | 2 " " " " " 40 " " | 0.5 |
| 18 R | 6 | 1 : 1½ : 3 K | 2 " " " " " " " " | 0.5 |

SLABS WITH ROD REINFORCING. DRY SUB-GRADE.

* K = Cement concrete.

3.—In general, plain concrete slabs show no more resistance to impact delivered at the edge than to impact delivered at the corner.

4.—Transverse cracks, and longitudinal cracks near the sides of a road slab, may be caused by impacts delivered at the edge of the slab.

5.—Plain concrete of 1 : 3 : 6 mix shows from about 60 to about 80% of the resistance to impact of plain concrete of 1 : 1½ : 3 mix. The lean mix also shows more variation in strength.

6.—Reinforcing steel in concrete slabs, when present in sufficient quantity, and when placed so as to receive tensile stress, adds to the resistance of the slab to impact.

7.—After a crack occurs, small rods closely spaced seem to be more effective than the same percentage of steel, in large rods, widely spaced.

8.—There is very little evidence of "cushioning" due to bituminous tops at temperatures of 32° cent. (90° Fahr.), or less.

9.—Bituminous tops do not seem to add to the slab strength of a concrete base with the possible exception of the 4 and 6-in. bases on a dry sub-grade.

## PITTSBURG, CALIF., TEST ROAD

Brief mention should be made of the results of the service tests made on the specially constructed road at Pittsburg, Calif. This report has already been published in detail.* The test was initiated by the Columbia Steel Company of Pittsburg, and, later, became a co-operative investigation by the California Department of Public Works and the U. S. Bureau of Public Roads.

* Report of Highway Research at Pittsburg, Calif., 1921–22, Dept. of Public Works, California.

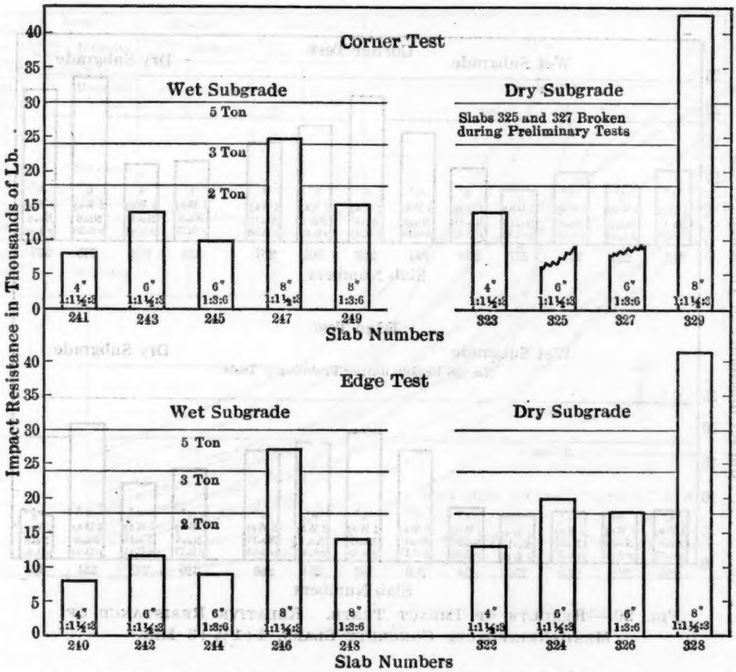



FIG. 18.—RESULTS OF IMPACT TESTS, RELATIVE RESISTANCE OF PLAIN CONCRETE SLABS.

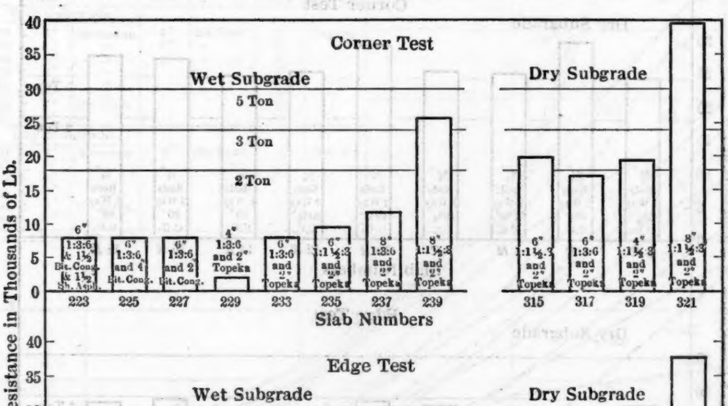



FIG. 19.—RESULTS OF IMPACT TESTS. RELATIVE RESISTANCE OF CONCRETE SLABS WITH BITUMINOUS TOPS.

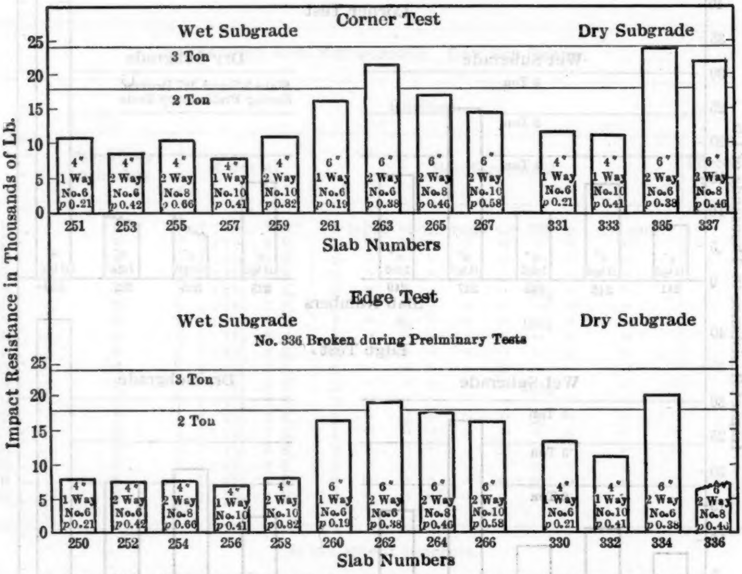



FIG. 20.—RESULTS OF IMPACT TESTS. RELATIVE RESISTANCE OF MESH-REINFORCED CONCRETE SLABS, 1 : 1½ : 3 MIX.

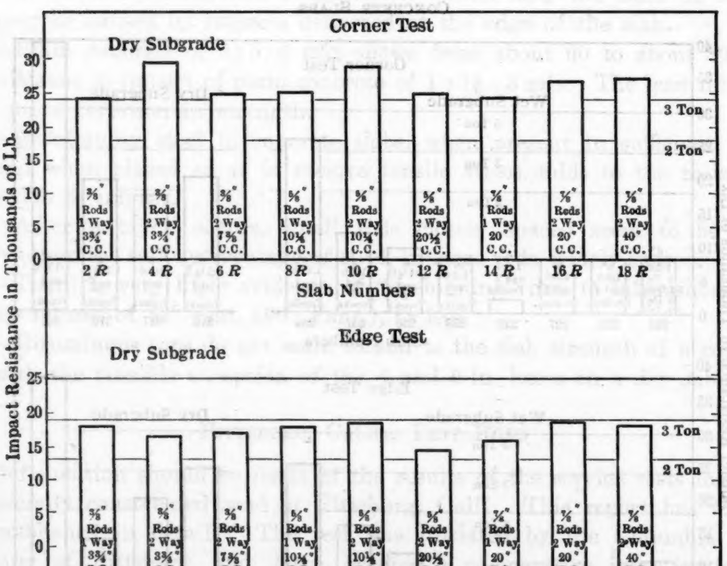
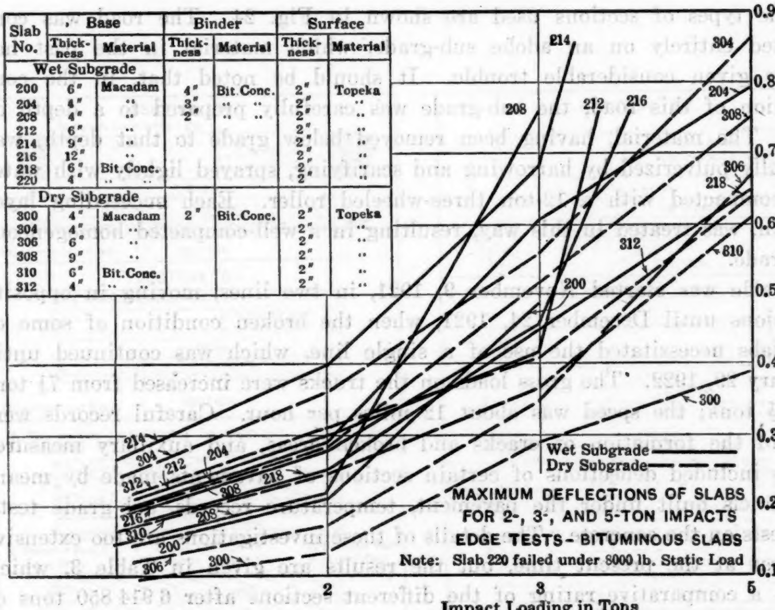
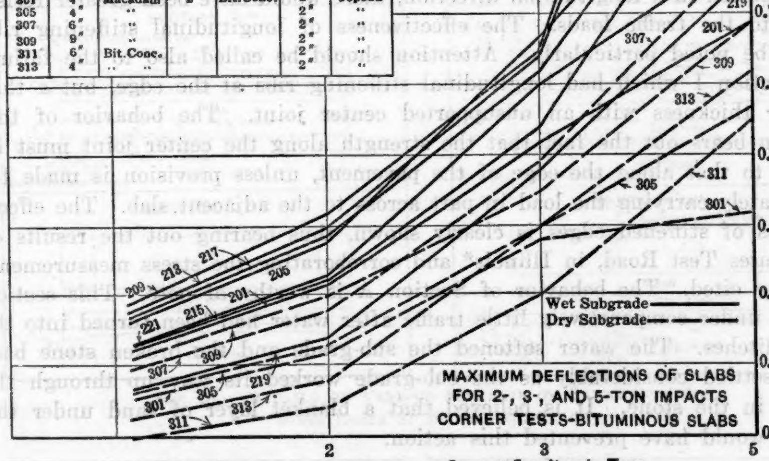



FIG. 21.—RESULTS OF IMPACT TESTS. RELATIVE RESISTANCE OF ROD-REINFORCED CONCRETE SLABS, 6 IN. THICK, 1 : 1½ : 3 MIX.



Slab No.
Base
Thick-ness
Material
Binder
Thick-ness
Material
Surface
Thick-ness
Material
Wet Subgrade
200 6" Macadam 4" Bit. Conc. 2" Topeka
204 4"
208 4"
212 6"
214 9"
216 12"
218 6" Bit. Conc.
220 4"
Dry Subgrade
300 4" Macadam 2" Bit. Conc. 2" Topeka
304 4"
306 6"
308 9"
310 6" Bit. Conc.
312 4"
Wet Subgrade
Dry Subgrade
MAXIMUM DEFLECTIONS OF SLABS
FOR 2-, 3-, AND 5-TON IMPACTS
EDGE TESTS - BITUMINOUS SLABS
Note: Slab 220 failed under 8000 lb. Static Load
Impact Loading in Tons
Maximum Deflection, in Inches


FIG. 22.



Slab No.
Base
Thick-ness
Material
Binder
Thick-ness
Material
Surface
Thick-ness
Material
Wet Subgrade
201 6" Macadam 4" Bit. Conc. 2" Topeka
205 4"
209 4"
213 6"
215 9"
217 12"
219 6" Bit. Conc.
221 4"
Dry Subgrade
301 4" Macadam 2" Bit. Conc. 2" Topeka
305 4"
307 6"
309 9"
311 6" Bit. Conc.
313 4"
Wet Subgrade
Dry Subgrade
MAXIMUM DEFLECTIONS OF SLABS
FOR 2-, 3-, AND 5-TON IMPACTS
CORNER TESTS-BITUMINOUS SLABS
Impact Loading in Tons
Maximum Deflection, in Inches


FIG. 23.

The types of sections used are shown in Fig. 24. The road was constructed entirely on an adobe sub-grade, which material in the past has always given considerable trouble. It should be noted that, in the construction of this road, the sub-grade was carefully prepared to a depth of 3 ft. The material, having been removed below grade to that depth, was carefully pulverized by harrowing and scarifying, sprayed lightly with water and compacted with a 12-ton three-wheeled roller. Each succeeding layer of 6 in. was treated in this way, resulting in a well-compacted homogeneous sub-grade.

Traffic was started November 9, 1921, in two lines, moving in opposite directions until December 24, 1921, when the broken condition of some of the slabs necessitated the use of a single line, which was continued until January 29, 1922. The gross loads on the trucks were increased from 7½ tons to 13½ tons; the speed was about 12 miles per hour. Careful records were kept of the formation of cracks and broken areas, and auxiliary measurements included deflections of certain sections of pavements made by means of tunnels built under the pavement, temperature records, sub-grade tests, and tests on the concrete. The details of these investigations are too extensive to treat at the present time, but the results are given in Table 3, which shows a comparative rating of the different sections after 6 914 850 tons of traffic had gone over the pavement. In arriving at the unit comparison of total computed cost per mile, it was assumed that the cost of replacing broken areas of concrete would be three times that of laying the original concrete and that in all cases the expense of maintaining cracks would be 1 cent per lin. ft.

In general, it will be noted that the sections having a low percentage of failure are those which have been either stiffened or reinforced longitudinally. No doubt, if more of the transverse reinforcement in the various sections had been placed in a longitudinal direction, there would have been greater resistance to the traffic loads. The effectiveness of longitudinal stiffening ribs is to be noted particularly. Attention should be called also to the failure of Section I which had longitudinal stiffening ribs at the edge, but a thin center thickness with an unsupported center joint. The behavior of this section bears out the fact that the strength along the center joint must be equal to that along the edge of the pavement, unless provision is made for adequately carrying the load in part across to the adjacent slab. The effectiveness of stiffened edges is clearly shown, thus bearing out the results of the Bates Test Road, in Illinois* and corroborating the stress measurements already cited. The behavior of Section A is worthy of note. This section failed under comparatively little traffic after water had been turned into the side ditches. The water softened the sub-grade and the broken stone base then settled considerably as the sub-grade worked its way up through the voids in the stone. It is believed that a blanket layer of sand under the stone would have prevented this action.

* "Highway Research in Illinois", by Clifford Older, M. Am. Soc. C. E., *Proceedings*, Am. Soc. C. E., Papers and Discussions, February, 1924, p. 175.

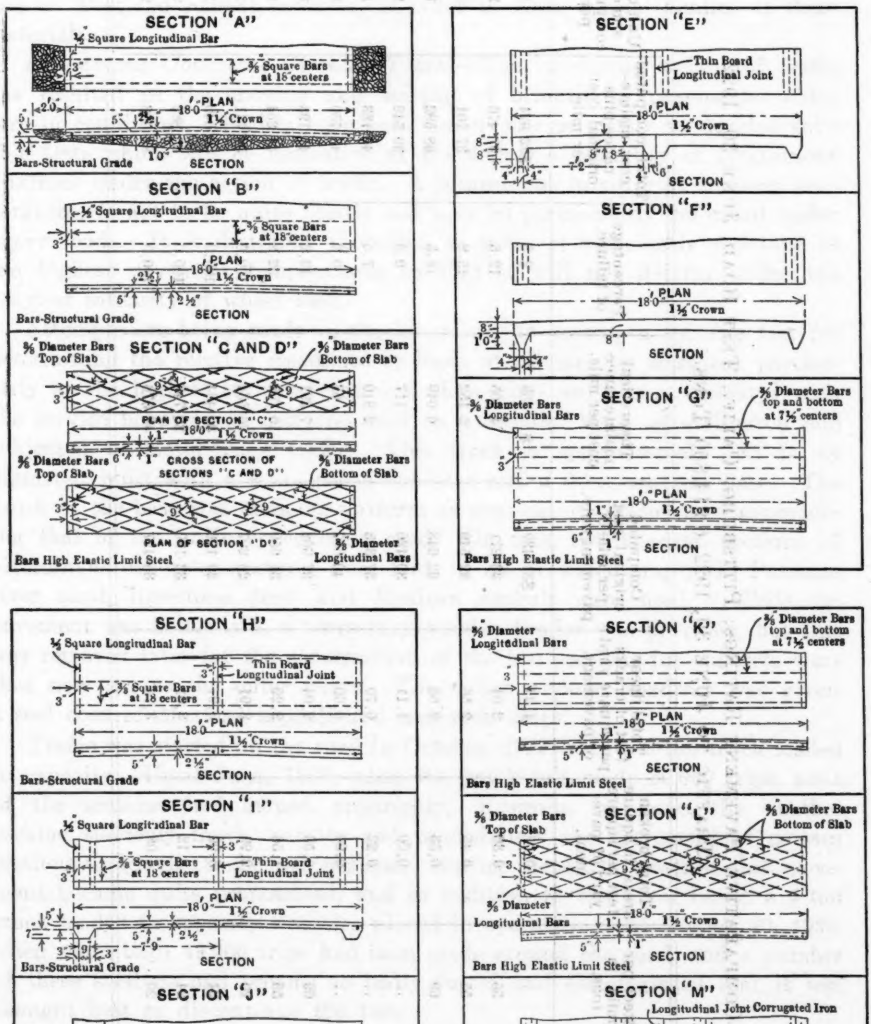



FIG. 24.—DETAILS OF VARIOUS SECTIONS OF PAVEMENTS FOR WOOD TESTS AT PITTSBURG, CALIF.

TABLE 3.—COMPARATIVE PAVEMENT RATINGS, PITTSBURG, CALIF., TEST ROAD, JUNE 30, 1922. TRAFFIC TO JUNE 30, 1922, 6 914 850 TONS.

| Section. | Concrete per mile as actually laid, in cubic yards. | Steel per mile, in tons. | Total broken areas, in square feet per square yard of pavement. | Total linear feet of cracks per square yard of pavement. | Computed original cost per mile of pavement only. | Theoretical maintenance cost per mile. | Percentage of failure. | Total computed cost, maintenance plus original cost per mile. | Unit comparison of total computed cost per mile. |
|---|---|---|---|---|---|---|---|---|---|
| J | 2 110 | .. | 0.07 | 1.44 | $31 439 | $ 885 | 2.8 | $32 324 | 1.000 |
| D | 1 845 | 55 | 0.36 | 2.01 | 33 271 | 4 205 | 12.6 | 37 476 | 1.159 |
| C | 1 798 | 55 | 0.45 | 2.56 | 32 549 | 5 152 | 15.8 | 37 701 | 1.166 |
| K | 1 722 | 69 | 0.53 | 2.66 | 32 643 | 6 049 | 18.5 | 38 692 | 1.197 |
| F | 2 719 | .. | 0.00 | 1.11 | 39 426 | 117 | 0.3 | 39 543 | 1.223 |
| M | 2 574 | .. | 0.22 | 0.70 | 37 323 | 2 810 | 7.5 | 40 133 | 1.242 |
| E | 2 806 | .. | 0.00 | 0.60 | 40 687 | 63 | 0.2 | 40 750 | 1.261 |
| G | 2 127 | 69 | 0.56 | 2.04 | 37 902 | 7 291 | 19.2 | 45 193 | 1.398 |
| L | 1 740 | 55 | 1.56 | 2.29 | 31 659 | 16 705 | 52.8 | 48 364 | 1.496 |
| B | 1 590 | 20 | 2.99 | 3.91 | 26 207 | 26 534 | 100.0 | 52 741 | 1.632 |
| H | 1 760 | 24 | 3.09 | 3.91 | 29 176 | 30 464 | 100.0 | 59 640 | 1.845 |
| I | 1 953 | 24 | 3.12 | 3.01 | 32 139 | 33 743 | 100.0 | 65 882 | 2.038 |
| A | 1 834 | 20 | 4.08 | 3.74 | 35 427 | 48 576 | 100.0 | 84 003 | 2.599 |

### Studies in Materials

Service tests have been conducted at Arlington on bituminous concrete and on Portland cement concrete, in order to make special studies of these materials.

*Bituminous Concrete—Circular Track.*—The increasing weight of traffic has resulted in the shoving and waving of bituminous pavements under certain conditions. Studies have been under way primarily to develop suitable tests which will be indicative of the degree of stability of bituminous mixtures under the action of traffic. A bituminous mixture at summer temperatures is apt to be quite plastic and may be permanently deformed under heavy loads. It is desirable, therefore, to make it sufficiently resistant at the highest pavement temperatures so that it will not deform under the heaviest intensity of wheel load.

Attempts are being made in the laboratory to develop a suitable test for determining the relative resistance to loads of bituminous mixtures, particularly at the highest temperatures, and these tests are being correlated with the service behavior of mixtures laid in a circular track at Arlington and subjected to motor-truck traffic. This track is approximately 185 ft. in diameter, built with a 6-in. gravel sub-base and a 6-in. concrete base. The finish of the base was made as uniform as possible, its smoothness approaching that of the ordinary concrete road. On this, twenty-seven sections of bituminous concrete mixtures were laid, in all of which trap-rock, Potomac river sand, limestone dust, and Mexican asphalt were used. While the pavement was being laid, a temporary wooden header was provided, but this was removed later for the construction of the test sections for concrete wear that now act as the outer curve. The entire circular roadway was given a seal coat of Mexican asphalt and trap-rock chips.

Traffic was started on the road in October, 1922, with a 3-ton truck loaded to capacity. Until June, 1923, when the truck had made 30 000 trips, none of the sections had moved, apparently. However, in June, the weather became suddenly much warmer and immediately the movement of certain sections under the traffic was noticed. During the warm weather, this movement became quite pronounced, and in addition to the 3-ton truck, a 5-ton truck loaded to capacity was also placed in operation until August 20, 1923, when more than 49 000 trips had been made around the road, and a number of these sections had become so badly rutted and disintegrated that it was thought best to discontinue the test.

During the last three stages, the average air temperature at 2 P. M., was, as follows:

| | |
|---|---|
| May 11 to June 25, 1923............ | 81° Fahr. |
| June 25 to July 20, 1923........... | 85.7° Fahr. |
| July 20 to August 17, 1923......... | 83.5° Fahr. |

From July 6 to August 16, temperatures were taken in one section at depths increasing by ½ in. Highest temperatures, about 2 P. M., were observed ½ in. below the surface, ranging from 95 to 132° Fahr., with a mean of 115° Fahr.

White lines were painted transversely across the pavement at the third-points of each section (Fig. 17). The diagrams, Fig. 25, give the maximum movement of these lines in the various sections. The greatest movement always occurred under the outer wheel, no doubt because of the increased wheel pressure produced by centrifugal force. It is hoped by the proper correlation of such service tests with laboratory studies that finally there will be evolved a suitable test for mixtures, which will indicate their degree of stability under various conditions of temperature and loading.

*Concrete Wear—Circular Track.*—In order to study the effect of a number of different factors on the wear of concrete pavements, a great many sections were built in a circular track, 200 ft. in diameter, immediately adjoining the bituminous track, previously described. It was subjected to the wear of a special machine (Fig. 17), equipped with solid rubber tires loaded to 600 lb. per in. of width. The driving was done through two of the rubber-tired wheels at a speed of approximately 20 miles per hour. The auxiliary steel wheels merely served to guide the apparatus. The first stage of the test consisted of 60 000 runs of this machine and the second stage of about 10 000 runs over a new path, but with two of the wheels equipped with skid chains. The results after 10 000 runs of the wearing device equipped with skid chains are shown in Fig. 26.

Data on the various sections used are given as follows, Groups I to VII, inclusive.

*Group I.—Quality of Crushed Stone as Coarse Aggregate.—*

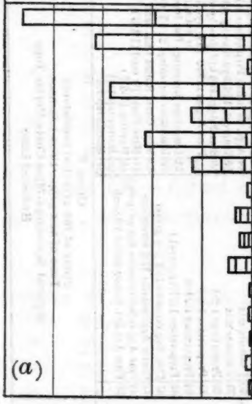
Mix.......................... 1:1½:3
Fine aggregate.............. Potomac River sand
Slump........................ 1 in.
Time of mix.................. 1 min.

Section 1.—Martinsburg limestone, laid August 14, 1922
" 2.—Bound Brook, N. J., trap, laid August 14, 1922
" 3.—Sioux Falls, S. Dak., quartzite, laid August 15, 1922
" 4.—Llano, Tex., granite, laid August 15, 1922
" 5.—Burnet County, Tex., limestone, laid August 15, 1922
" 6.—Sandstone County, Minn., sandstone, laid August 15, 1922
" 7.—Esserville, Va., Sandstone "B", laid August 15, 1922
" 8.—Bellevue, Ohio, limestone, laid August 16, 1922
" 9.—Rogers, Mich., limestone, laid August 16, 1922
" 10.—Stone City, Iowa, limestone, laid August 16, 1922

*Group II.—Quality of Gravel as Coarse Aggregate.—*

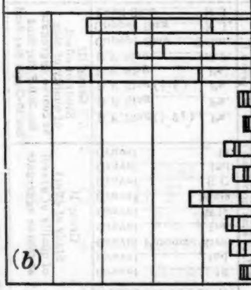
Mix.......................... 1:1½:3
Fine aggregate.............. Potomac River sand
Slump........................ 1 in.
Time of mix.................. 1 min.

Section 11.—Gravel from Potomac River, laid August 16, 1922
" 12.— " " Plainfield, N. J., laid August 16, 1922
" 13.— " " Williamsport, Ind., laid August 17, 1922
" 14.— " " South Bend, Ind., laid August 17, 1922
" 15.— " " Wausau, Wis., laid August 17, 1922
" 16.— " " Ludlow, Mass., laid August 17, 1922
" 17.— " " Wateree, S. C., laid August 17, 1922
" 18.— " " Evansville, Ind., laid August 18, 1922
" 19.— " " Hancock, Iowa, laid August 18, 1922



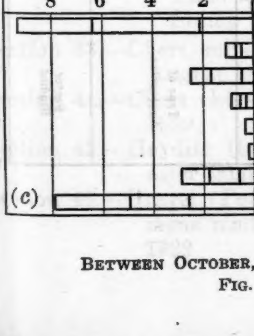

| MAXIMUM MOVEMENT | ASPHALT | | GRADING OF TOTAL AGGREGATE | | | | | | | | SEC. No. | MIX. No. |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 80″ 60″ 40″ 20″ | Pen | % | R 1¼ | R ¾ | ¾-¼ | ¼-10 | 10-40 | 40-80 | 80-200 | P-200 | | |
| | 86 | 4.5 | 1.2 | 20.5 | 45.4 | 9.5 | 8.6 | 6.6 | 5.3 | 4.1 | 17 | 1 |
| | 76 | | | | | | | | | | 18 | 2 |
| | 65 | | | | | | | | | | 19 | 3 |
| | 55.5 | | | | | | | | | | 16 | 4 |
| | 75 | 4.9 | 1.6 | 23.6 | 34.0 | 9.9 | 13.0 | 9.8 | 6.5 | 3.2 | 13 | 5 |
| | 64.5 | | | | | | | | | | 14 | 6 |
| | 55.5 | | | | | | | | | | 12 | 7 |
| | 46 | | | | | | | | | | 15 | 8 |
| | 75.5 | 6.0 | 1.5 | 18.7 | 31.6 | 7.0 | 16.5 | 13.8 | 8.7 | 3.7 | 3 | 9 |
| | 65 | | | | | | | | | | 2 | 10 |
| | 55 | | | | | | | | | | 1 | 11 |
| | 45.5 | | | | | | | | | | 4 | 12 |
| | 76 | 6.3 | | 9.6 | 23.8 | 9.4 | 23.5 | 19.0 | 10.6 | 4.1 | 23 | 13 |
| | 65 | | | | | | | | | | 21 | 14 |
| | 55 | | | | | | | | | | 20 | 15 |
| | 45 | | | | | | | | | | 22 | 16 |

BETWEEN OCTOBER, 1922, AND JUNE 25, 1923, A TOTAL OF 37 414 RUNS.




| MAXIMUM MOVEMENT | ASPHALT | | GRADING OF TOTAL AGGREGATE | | | | | | | | SEC. No. | MIX. No. |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 8″ 6″ 4″ 2″ | Pen | % | R 1¼ | R ¾ | ¾-¼ | ¼-10 | 10-40 | 40-80 | 80-200 | P-200 | | |
| | 75.5 | 6.0 | 1.5 | 18.7 | 31.6 | 7.0 | 16.5 | 13.8 | 8.7 | 3.7 | 3 | 9 |
| | 65 | | | | | | | | | | 2 | 10 |
| | 55 | | | | | | | | | | 1 | 11 |
| | 45.5 | | | | | | | | | | 4 | 12 |
| | 75.5 | 5.2 | 6.9 | 26.0 | 25.7 | 11.7 | 15.5 | 10.8 | 7.0 | 3.3 | 6 | 17 |
| | 55 | | | | | | | | | | 7 | 18 |
| | 45.5 | | | | | | | | | | 5 | 19 |
| | 55.5 | 6.3 | | 16.4 | 30.4 | 12.3 | 19.0 | 12.5 | 6.7 | 2.7 | 11 | 20 |
| | 75 | 6.0 | | 18.2 | 27.5 | 7.7 | 18.0 | 15.2 | 9.6 | 3.8 | 9 | 21 |
| | 55.5 | | | | | | | | | | 8 | 22 |
| | 46 | | | | | | | | | | 10 | 23 |

BETWEEN OCTOBER, 1922, AND JULY 23, 1923, A TOTAL OF 44 197 RUNS.




| MAXIMUM MOVEMENT | ASPHALT | | GRADING OF TOTAL AGGREGATE | | | | | | | | SEC. NO. | MIX. NO. |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 8″ 6″ 4″ 2″ | Pen | % | R1¼ | R¾ | ¾-¼ | ¼-10 | 10-40 | 40-80 | 80-200 | P200 | | |
| | 55 | 6.0 | 1.5 | 18.7 | 31.6 | 7.0 | 16.5 | 13.8 | 8.7 | 3.7 | 1 | 11 |
| | 55 | 5.2 | 6.9 | 26.0 | 25.7 | 11.7 | 15.5 | 10.8 | 7.0 | 3.3 | 7 | 18 |
| | 55 | 6.3 | | 16.4 | 30.4 | 12.3 | 19.0 | 12.5 | 6.7 | 2.7 | 11 | 20 |
| | 55.5 | 6.0 | | 18.2 | 27.5 | 7.7 | 18.0 | 15.2 | 9.6 | 3.8 | 8 | 22 |
| | 55 | 4.9 | | 20.5 | 24.8 | 9.9 | 21.8 | 16.1 | 4.7 | 2.2 | 25 | 24 |
| | 55 | 5.0 | 4.7 | 24.6 | 23.5 | 12.4 | 19.1 | 13.6 | 4.3 | 2.5 | 24 | 25 |
| | 55 | 7.5 | 4.3 | 11.9 | 28.7 | 9.5 | 11.9 | 15.1 | 13.6 | 9.3 | 26 | 26 |
| | 55 | 7.3 | | 9.5 | 31.6 | 5.4 | 7.6 | 18.8 | 18.6 | 8.5 | 27 | 27 |

BETWEEN OCTOBER, 1922, AND AUGUST 20, 1923, A TOTAL OF 49 317 RUNS.

FIG. 25.—SHORING OF ASPHALT PAVEMENTS.

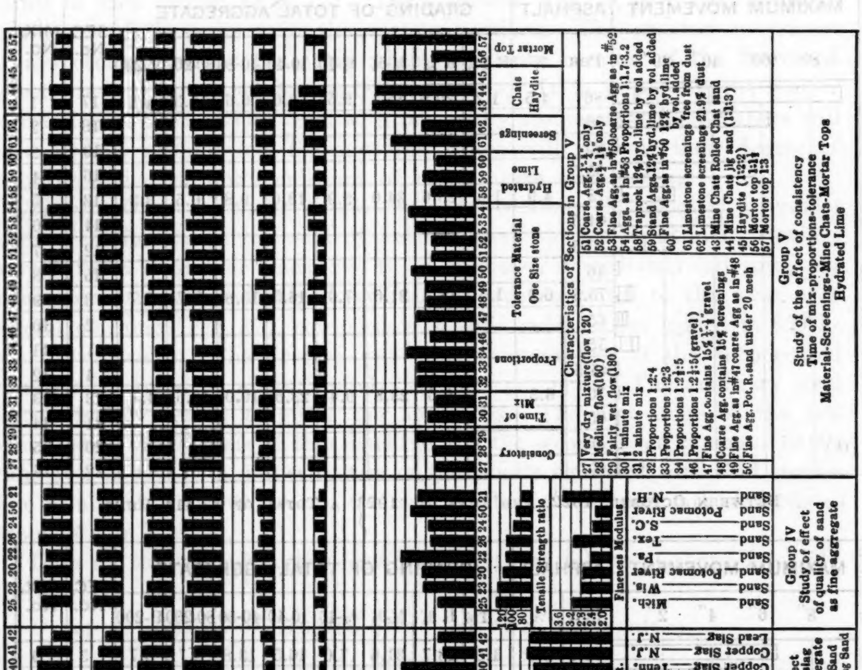



FIG. 26.—CHART OF RESULTS OF WEAR TESTS, CONCRETE PAVEMENTS, AFTER 10 000 RUNS, WHEELS EQUIPPED WITH SKID CHAINS.

*Group III.—Quality of Sand as Fine Aggregate.—*

Mix.......1 : $1\frac{1}{2}$ : 3, using $1\frac{1}{2}$-in. = $\frac{1}{4}$-in. Martinsburg limestone
Slump......................1 in.
Time of mix..............1 min.

Section 20.—Sand from Potomac River, laid August 18, 1922
" 21.— " " Concord, N. H., laid August 18, 1922
" 22.— " " Point Marion, Pa., laid August 18, 1922
" 23.— " " Wausau, Wis., laid August 21, 1922
" 24.— " " Pon Pon, S. C., laid August 21, 1922
" 25.— " " St. Clair River, Mich., laid August 21, 1922
" 26.— " " Colorado River, Texas, laid August 21, 1922

*Group IV.—Effect of Consistency, Time of Mix, and Proportioning.—*

Proportions: As indicated, using Martinsburg limestone $1\frac{1}{2}$ in. — $\frac{1}{4}$ in. as coarse aggregate and Potomac River sand as fine aggregate.

Section 27.—1 : $1\frac{1}{2}$ : 3 mix...$\frac{1}{2}$-in. slump...1-min. mix, laid August 21, 1922
" 28.— " " ...1- " " ...1- " " laid August 23, 1922
" 29.— " " ...2- " " ...1- " " "
" 30.— " " ...1- " " ...$\frac{1}{2}$- " " "
" 31.— " " ...1- " " ...2- " " "
" 32.—1 : 2 : 4 " ...1- " " ...1- " " "
" 33.—1 : 2 : 3 " ...1- " " ...1- " " "
" 34.—1 : $2\frac{1}{2}$ : 5 " ...1- " " ...1- " " laid August 25, 1922

*Group V.—Quality of Slag as a Coarse and Fine Aggregate.—*

Proportions: 1 : $1\frac{1}{2}$ : 3 by volume, using Potomac River aggregate, except where otherwise indicated; slump, 1 in.; time of mix, 1 min.

Section 35.—B. F. slag, Pottstown, Pa., $2\frac{1}{2}$ in.-$\frac{1}{4}$ in., laid August 25, 1922
" 36.— " " " $1\frac{1}{2}$ " -$\frac{1}{4}$ " "
" 37.— " " " $\frac{3}{4}$ " -$\frac{1}{4}$ " "
" 38.— " " " $1\frac{1}{2}$ " -$\frac{1}{4}$ " slag sand, laid August 25, 1922

Section 39.—B. F. slag, Wharton, N. J., $1\frac{1}{2}$ in.-$\frac{1}{4}$ in., hand-picked, laid August 25, 1922

Section 40.—Copper slag, Copperhill, Tenn., $1\frac{1}{2}$ in.-$\frac{1}{4}$ in., slag sand, laid August 28, 1922

Section 41.—Copper slag, Perth Amboy, N. J., $1\frac{1}{2}$ in.-$\frac{1}{4}$ in., laid August 28, 1922

Section 42.—Lead slag, Perth Amboy, N. J., $1\frac{1}{2}$ in.-$\frac{1}{4}$ in., laid August 28, 1922

*Group VI.—Quality of Miscellaneous Aggregates.—*

Proportions ............As indicated.
Time of mix............1 min.
Slump ....................1 in.

Section 43.—Chert chips ($\frac{1}{2}$ in.-$\frac{1}{4}$ in.) rolled chat sand, 1 : $1\frac{1}{2}$ : 3 mix, laid August 28, 1922

Section 44.—Chert chips (mine-run) jig sand, 1 : 1 : 3 mix, laid August 28, 1922

Section 45.—Haydite (burnt clay) coarse and fine, mix changed to 1 : 2 : 2 after trial batch, laid August 28, 1922

Section 46.—Gravel (Potomac) and Potomac sand, 1 : $2\frac{1}{2}$ : 5 mix, only specimens made were three compression cylinders, laid August 28, 1922

*Group VII.—Effect of Tolerance Material, One-Size Stone, One-Size Sand, Screenings in Place of Sand, Mortar Tops, and Hydrated Lime.—*

Proportions .......1 : 1½ : 3, except as noted.
Time of mix........1 min.
Slump ............1 in.

Section 47.—Martinsburg limestone, Potomac River sand, with 15% gravel, ½-¼ in., laid August 29, 1922

" 48.—Martinsburg limestone, with 15% screenings free from dust, Potomac River sand, laid August 29, 1922

" 49.—Martinsburg limestone, with 15% screenings free from dust, Potomac River sand, laid August 29, 1922

" 50.—Martinsburg limestone, Potomac River building sand, laid August 30, 1922

" 51.—Martinsburg limestone, ¼ in.-¾ in. material only, Potomac River sand, laid August 30, 1922

" 52.—Martinsburg limestone, ¾ in.-1½ in. material only, Potomac River sand, laid August 30, 1922

" 53.—Martinsburg limestone, ¾ in.-1½ in. material only, Potomac River building sand, laid August 30, 1922

" 54.—Martinsburg limestone, ¾ in.-1½ in. material only, Potomac River building sand, 1 : 1.7 : 3.2, laid August 30, 1922.

" 55.—Martinsburg limestone, Martinsburg limestone screenings, $\frac{1}{16}$ in.-¼ in. material only, laid August 30, 1922

" 56.—Mortar top, Potomac River sand, 1 : 1½, laid August 31, 1922

" 57.—Mortar top, Potomac River sand, 1 : 3, laid August 31, 1922

" 58.—Trap-rock dresser, Junction, Wis., Potomac River sand, with 12% (by volume) hydrated lime, laid August 31, 1922

" 59.—Martinsburg limestone, Potomac River sand, with 12% (by volume) hydrated lime, laid August 31, 1922

" 60.—Martinsburg limestone, Potomac River building sand, with 12% (by volume) hydrated lime, laid August 31, 1922

" 61.—Martinsburg limestone, Martinsburg limestone screenings, free from dust (4%), laid September 1, 1922

" 62.—Martinsburg limestone, Martinsburg limestone screenings, 21.9% dust, laid September 1, 1922

## Miscellaneous Investigations

In addition to the investigations previously described, a number of other investigations are under way in the Bureau of Public Roads, or in various other institutions in co-operation with the Bureau, covering different phases of the design of highways. For example, the University of Maryland is determining the elastic properties of concrete under repeated stress and is likewise making a study of Poisson's ratio for concrete. The first study will furnish results of value in connection with the selection of a proper unit stress and the second, results which may be used after the development of the proper design theory.

Similar studies are being made at Purdue University relating to fatigue tests of concrete and also the stress produced in concrete by moisture changes.

The University of Georgia is making some special studies of sand-clay and top-soil roads.

The Iowa State College is working on problems of highway bridge impact and loads on highway culverts.

Studies are also being made by other organizations, in co-operation with the Bureau of Public Roads, dealing with other phases of road design. It is believed that the data now being obtained will finally result in the use of more scientific and more rational methods of road design than have been found possible in the past.

# SANITATION—ITS RELATION TO HEALTH AND LIFE*

By George C. Whipple,† M. Am. Soc. C. E.

In an able address‡ before the Sanitary Engineering Division of the Society on January 16, 1923, Harrison P. Eddy, M. Am. Soc. C. E., described the present status of sanitary engineering and outlined its scopes and activities. The speaker wishes to present an even more fundamental aspect of sanitation, namely, its relation to health and life.

The subject cannot be discussed logically without first considering some of the words commonly used in connection with it. What is meant by health? This word comes from the old Anglo-Saxon, "haelth". There are several allied words, such as "hale", "whole", "wholesome", and "holy". This idea of wholeness seems to be fundamentally involved. Health applies to the whole body. The common expression, "all right", gives the proper idea. Health is bodily "all-right-ness".

The word, health, is used in two ways: Absolute and relative. It may refer to a state of physical and mental wholeness to the exclusion of some lower state, that is, health may mean normal health or good health; or it may represent a designated relation to such a normal state, as when one says excellent health, bad health, or some other variety of health. Giving the word its full meaning, it is possible to analyze it, or, at least, to state its concepts.

Health implies organic soundness. It is stated in the Bible that "they that are whole need not a physician * * *." It implies a condition in which all the organs function naturally and the body as a whole operates effectively. It implies a condition in which the body responds adequately to its environment and in which there exists a sense of comfort, or physical contentment. It implies also mental soundness and mental activity. In other words, full natural health has four elements: A sound mind, a sound body, natural functioning, and fitting responses to outside influences during a considerable period of time.

It is not surprising that some people should pick out one of these concepts and emphasize it to the neglect of the others, thus giving to the word, health, a limited or circumscribed meaning. Some limit the word to the soundness of the physical body, some exclude the senses, some overlook the nervous system and mental state. To many physicians, and to some health officers, health means merely the absence of disease. The increasing use of vital statistics, based on deaths and cases of sickness as measures of the healthiness of a community, is tending to spread this unfortunate partial use of the word. The

* Presented before the Sanitary Engineering Division, January 15, 1924.

† Cons. Engr.; Prof. of San. Eng., Harvard Univ., Cambridge, Mass.

‡ *Proceedings*, Am. Soc. C. E., January, 1924, Papers and Discussions, p. 3.

positive elements of health are indeed more difficult to measure than the negative departures from good health; but, nevertheless, there is a positive side which deserves consideration. There is a condition in which vitality is being stored up, just as there is a condition in which it is being exhausted. Some day, perhaps, physiologists and psychologists will find better ways to measure and describe health. As an old student of vital statistics, the speaker will state that death rates are an inadequate measure of the health of a community. Statistics of sickness are available only for certain major diseases, whereas the total incapacitating influence of the more numerous minor ailments and disorders is probably greater than that of the diseases reported. One of the most difficult problems which vital statisticians have before them is that of finding and applying an adequate mathematical index of the health of a community. To define what is meant by normal health is difficult, because the normal is not the same at all times or in all places. Biometrics is a promising science; thus far, however, it has given little practical help to health officials.

### Environmental Complements to Health

One cannot get far in a discussion of health without taking account of environment; in fact, it is difficult to conceive of man apart from his environment. Normal health connotes a normal environment. Health is dependent in great measure on inborn, constitutional, hereditary factors. Length of life depends more on these intrinsic forces than on anything else. The environmental factors, however, are of especial interest to sanitary engineers. They are more readily controlled than the personal and hereditary factors. Some environmental factors injure health; others promote it. Omitting the constitutional phases of the subject, three classes of injurious environmental factors may be distinguished:

1.—There are damaging factors, which include infections, poisonings, and accidents. The biological, chemical, and physical injuries inflicted by them tend to drag health downward. Whatever benefits they offer are only by way of warding off worse injuries.

2.—There are what may be called functional factors, which have to do with breathing, eating and drinking, excretion, sleep and exercise, body temperature, and growth. Depending on their character, they affect the body either for good or for ill; they may injure health or promote it; they are either negative or positive. Good food may promote health; poor food may injure health. Good food, pure water, fresh air, proper lighting, and exercise tend to create a store of vitality; poor food, impure water, foul air, inadequate lighting, and fatigue tend to deplete the health reserves. These matters to a large extent come under the head of hygiene.

3.—There are also sensatory factors, that is, environmental influences which affect the perceptions and, through them, the bodily functions. These also are positive or negative. Bad smells and tastes are injurious; whereas agreeable odors and tastes are beneficial. Excessive noise is injurious; music is beneficial. Sights of ugliness and cruelty are injurious; beauty is a great uplifting force. In these matters, the mental and the physiological are so intertwined that they cannot be kept apart by definition and it is hardly worth

while to try to do so, if health is given the broad interpretation understood by the man on the street.

Of the three kinds of environmental influences, those which damage the health by infection, poisoning, or physical injury are of major importance. Exclusive of infections, the effect of the quality of air, water, and food on human health is relatively slight. The sensatory or perceptional factors are of even less importance. Nevertheless, these minor factors which injure health indirectly should be recognized and their influence not denied because they cannot be evaluated comparatively. Sometimes their effect is great.

Fig. 1 indicates the three classes of factors and their relative importance. The lines are not drawn to any scale, because the required data do not exist.

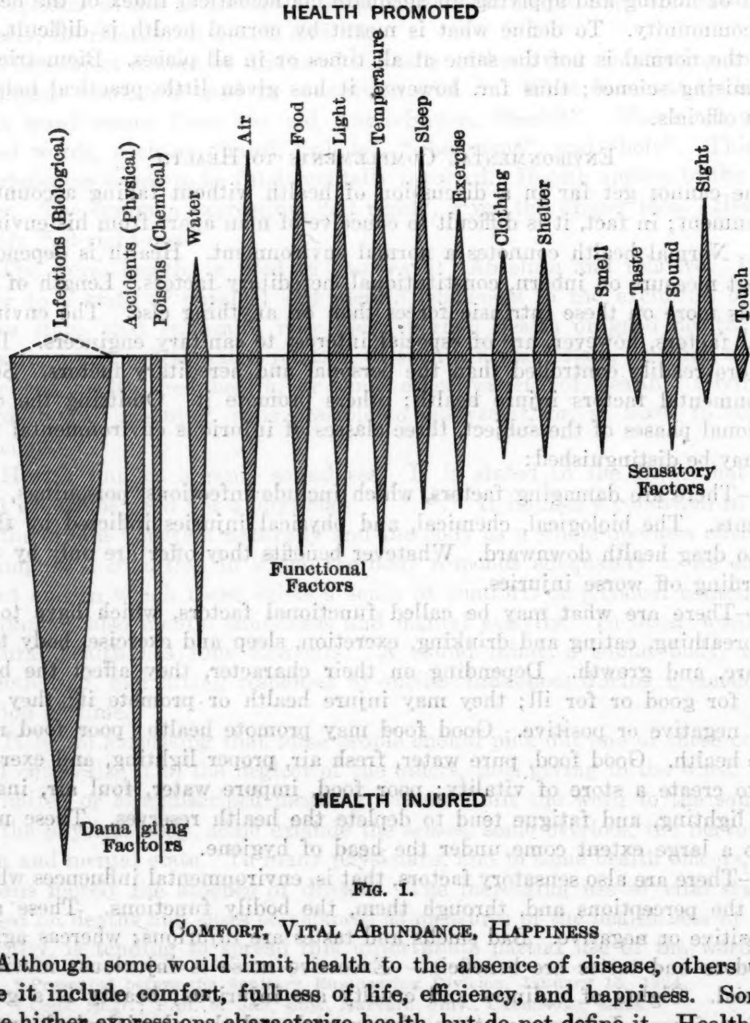



FIG. 1.

### COMFORT, VITAL ABUNDANCE, HAPPINESS

Although some would limit health to the absence of disease, others would have it include comfort, fullness of life, efficiency, and happiness. Some of these higher expressions characterize health, but do not define it. Health leads to happiness, but health is not happiness.

In his recent book on "Personal Hygiene Applied", Dr. J. F. Williams, of Columbia University, has defined health as "the quality of life that renders the individual fit to live most and to serve best." Here, emphasis is placed on the motive for health rather than on health itself. The word, comfort, seems to be so closely related to health as to be inseparable from it. Both in popular writings and in laws, the words, "health and comfort of the people", are found. It is significant that the laws do not say health or comfort. The ideas are conjoined. The senses are closely related both to the physiological and the mental. Through them, one experiences physical comfort and mental contentment, or physical discomfort and mental dissatisfaction.

Furthermore, it must be admitted that just as the body, its functional activities, and its sensations, may affect the mental state, so the mental state may affect the functions of the body. The two are not antagonistic, but complementary, and perfect health results from what Carlisle called their "sweet co-operation."

## Nuisances and Annoyances

When the environmental factors previously mentioned decrease in magnitude and duration, their influence on health may be so small as to be practically negligible. They become mere annoyances or inconveniences. A slight cold, a sliver in the finger, the occasional loss of sleep, a fleeting unpleasant odor may annoy without doing actual injury to health. Obviously, there are quantitative factors and time factors which should be recognized. People differ as to their sensitiveness to environmental factors. A mere annoyance to one person may be actual injury to another.

The word, nuisance, is so largely a legal term that it is best not to use it in any other sense. A nuisance may be a mere annoyance, a discomfort; or it may be an injury to health, safety, or morals. Blackstone said that a "nuisance is anything that worketh hurt, inconvenience, or damage". Modern legal thought adds the ideas of substantial damage and unreasonable use of property and liberty. A nuisance is a question of fact. As Jeremiah Smith stated, "A nuisance is whatever the Court says is a nuisance", and, of course, the jury must base its verdict on the evidence in the particular case. There are, to be sure, statutory nuisances. There are some things which are considered to be nuisances *in esse*. For example, in some places, the mere emission of black smoke from a chimney is declared by statute to be a nuisance; hence, in a particular case in Court, it is necessary to prove the existence of the smoke, not that the black smoke in question injures the health and safety or annoys people.

## Public Health

Public health is usually understood in a general and vague way to mean the collective health of the people in a given community. It is measured negatively and expressed very crudely in terms of birth rates, death rates, sickness rates, and the like, little account being taken of what may be called the minor diseases and departures from health and no account being taken of any of the positive elements of health. Although no adequate definition of

public health has been formulated, the term has grown in popularity until it has become almost a fetish. It is replacing the word, health, in unreasonable ways. State Boards of Health are becoming Departments of Public Health. Schools affording training in this branch are called "Schools of Public Health", whereas the term, Health School, would probably be better. The adjective, "public", limits the word, "health". Health is more inclusive than public health. The International List of the Causes of Death contains the titles of more than two hundred diseases, all, of course, are injurious to health; but of these diseases only about fifty are commonly declared by health authorities to be dangerous to the public health. These are chiefly the more serious of the communicable diseases. Yet, at present, many of the newer activities of health departments are being devoted to what would seem to be individual health rather than public health. Without questioning the desirability of protecting and even promoting individual health, there is a serious question as to how far it is just, wise, and economical to spend public moneys for individual benefits. Some argue that whatever affects the death rate affects the public health. This is not a sound argument, for the death rate itself is a composite of the death rates from many causes, some of which have a public significance and some of which have not. Some argue that the health of every individual has an economic bearing on the whole social community. Some say that, whereas in rural communities individual health remains individual, in a large city the concentration of people causes all health to be public health. It is somewhat anomalous that at the same time that the broad term, health, is being superseded by the narrower term, public health, in governmental administration, the functions of health departments are being extended.

When the environmental factors which affect the public health are considered, the uncertainties are still greater. Infections, such as typhoid fever, which are broadcasted through polluted water or milk, unquestionably affect the public health; and so do infections which form a continuous chain from person to person, as in the case of common colds, venereal diseases, and infections which are spread by insects. There are infections, however, which are essentially personal and which do not to any great extent spread from person to person. There are some poisonous factors and accident hazards which are likely to affect many people and there are some which can affect only a few. The factors which influence functional activities and the senses affect individuals more often than large groups of people, but a factory which spreads irritating fumes to such an extent that many people are functionally injured through their senses, may fairly be said to injure the public health.

This distinction between injuries to health and injuries to the public health would be of great importance if it were not for the fact that the police power of the State, the common law principle under which Governments act, is not limited to public health, but deals with health. The distinction between benefits to health and benefits to public health is important, because it involves justice in taxation.

### Public Nuisances

Although a nuisance is something for the Courts to decide, public nuisances have sometimes been given a statutory basis. In New York State, the Penal Code defines a public nuisance thus:

"A public nuisance is a crime against the order and economy of the State and consists of unlawfully doing an act, or omitting to perform a duty, which act or omission:

"1. Annoys, injures, or endangers the comfort, repose, health or safety of any considerable number of persons; or
"2. Offends public decency; or
"3. Unlawfully interferes with, obstructs, or tends to obstruct, renders dangerous for passage, a lake or a navigable river, bay, stream, canal, or basin, or a stream, creek or other body of water which has been dredged or cleared at public expense, or a public park, square, street or highway; or
"4. In any way renders a considerable number of persons insecure in life or the use of property."

It is interesting to note that the words, safety, health, repose, and comfort, of this statute are grouped together and form a sequence not unlike that of the words, damaging, functional, and sensatory factors, already described, whereas decency is set apart, perhaps, as a moral factor.

In deciding whether something, as, for example, a disagreeable smell, is a public nuisance, it is necessary to consider its magnitude, distance, frequency, and duration of occurrence, incongruity as to time and place, its cause with reference to unreasonable use of property or liberty, and the number, character, and condition of the people affected.

It might be justice for the Court to ignore a strong odor if very rare, or a more frequent odor if mild; to ignore a certain odor in an industrial district, but not in a residential district; to ignore an odor that prevails in the day time, but not at night; to ignore an odor in a sparsely settled region, but not in a populous region. Consideration should also be given to the effect of the alleged nuisance on persons in feeble health as well as on those in ordinary good health.

### Sanitation and Public Sanitation

Sanitation is the art of securing and maintaining a clean environment for the protection and promotion of health and comfort. In common parlance, emphasis is placed on this idea of cleanliness. Things which have to do with safety to life and limb, such as safety appliances against accident or fire, are usually not included under sanitation; but, on the other hand, mosquito suppression is included, even if it is not strictly a matter of cleanliness.

Those parts of sanitation which relate to communities rather than to individual persons and houses, come under the head of Sanitary Engineering, a branch of Civil Engineering. Sanitary engineers now deal with such matters as: (1) Water supply and water purification; (2) plumbing; (3) sewerage, sewage disposal, and river cleaning; (4) disposal of industrial wastes; (5) street cleaning and the collection and disposal of municipal wastes; (6) drainage and mosquito control; and (7) air supply and purification.

In some respects, the sanitation of buildings is a question of architecture rather than of engineering. At any rate, there are in architecture many problems of plumbing, heating, ventilation, lighting, basement drainage, and the orientation of buildings with reference to sunlight, all of which have important relations to health.

It may be worth while to consider how the methods and equipment of the art of sanitation affect health.

Water purification protects against damage by infection and lead poisoning, and against unpleasant tastes and odors. The quality of water is also a functional and sensatory factor.

Sewage treatment, river cleaning, and the disposal of industrial wastes are less important than water purification as far as damage by infection or poison is concerned, but, as sensatory factors, they are very important.

Faulty plumbing is only a slight damaging factor, infections, poisons, and accidents being very infrequent. It may, however, affect the senses of smell, sight, or sound, and may have an influence on the quality of the air, hence on breathing. Good plumbing promotes health by encouraging the use of water.

Inadequate heating and ventilation are chiefly functional factors, involving breathing, temperature control, sleeping, exercise, elimination of water, and growth. They are also sensatory and damaging factors.

Garbage disposal is of very little importance as far as infection is concerned, but has value as a sensatory and functional factor.

Street cleaning involves problems of damage by infection and accident, but the sensatory and functional elements are more important. The same is true of other phases of air sanitation, such as smoke control.

Mosquito control, in some parts of the country, is an important factor in infection; everywhere, it is a sensatory and functional factor.

The branches of sanitation which have to do with housing, namely, heating and ventilation, plumbing, lighting, artificial illumination, and noise prevention, attain their greatest importance as functional factors, although problems of infection, accident, poisons, and the senses may be involved. The more the speaker studies the subject, the more he is impressed with the supreme importance of what is called "housing" on human health. Housing is referred to in its broad aspect, from city planning and the regulation of building on the one hand to house-keeping on the other. The effect of housing is unlike that of the sudden infection of a water supply—spectacular and fearsome—it is slow and insidious and influences health and growth through the senses and the functions. Mathematically, this relation is obscure, illusive, and intermixed with the factors of poverty and ignorance, yet long experience shows that it exists. Rat-proofing is a phase of the housing problem which has a distinct infectional relation.

Industrial sanitation has problems which include many of the foregoing. Accidents, poisonings, and infections predominate, but there are also functional and sensatory factors.

Food sanitation involves important problems of infection, poisoning, and functions; whereas clothing sanitation has a functional bearing.

No attempt has been made to analyze these activities carefully or in detail, but they deserve thought and are often brought in question in the Courts.

An important point is that the object of sanitation is not merely to prevent injury to health, although it is largely that, but also to promote health. Water is filtered to make it clean, not merely to remove the germs of disease. Clean water is health promotive, whereas a chlorinated dirty water is merely health protective. Sunlight and pure air in abundance are emphatically health promotive agencies. These positive benefits of sanitation are too often overlooked. The speaker believes that there is a moral power in cleanliness; and that good sanitation contributes not only to the elimination of disease and positive betterment of health, but to those elements of life which make for better men and women and a higher type of civilization.

There is a tendency on the part of some health officials to belittle sanitation, to emphasize the infections in their personal relations, and to ignore the importance of environmental conditions apart from infections. From the standpoint of disease they are right; from the standpoint of health, the speaker thinks they are wrong. It is probably true that overflowing cesspools, unclean yards and apartments, defective plumbing, dirty cellars, smelling garbage, and smoke, odors, and fumes from factories have a relatively small influence on the death rate as compared with the effect of the major infections which pass from person to person, or when compared with gross abuses of the laws of hygiene. Yet the speaker thinks it true that sanitation is still and always will be the foundation of sound public health. Before the days of Pasteur, an unclean environment was thought to be a direct cause of disease; it is now known that the effect is not direct, but indirect. Living organisms, not the dirt, are the active agents. Yet, the influence of dirt itself is there and must not be neglected. To give a single example, the incidence of tuberculosis among stone-cutters, has been found to be directly proportional to the proportion of silica in the stone-dust. American health workers in France have stated that it is almost impossible to instill into the minds of the people the importance of living hygienic lives, if in the cities in which they live no adequate attention is paid to the disposal of fecal matter, sink wastes, animal manure, and household refuse. The speaker's students have written about their work and from everywhere the same story is told—sanitation is the foundation of health reform. We cannot have a sound mind in a sound body unless we have clean bodies in a clean world.

### Administration of Sanitation

It is important not to confuse the legal status of sanitation with its governmental administration. Granted that sanitation affects health and that even minor matters of sanitation may be justified as health measures, it does not necessarily follow that all sanitary activities should be carried out by the boards of health. Prior to the days of applied bacteriology, the control of sanitation was the traditional and almost the sole function of the board of health. In recent years, the emphasis has changed from the environment to the person, from sanitation to preventive medicine and hygiene. Furthermore, government itself is changing. Force is being supplemented by service. The func-

tions of local health boards and State health departments are accordingly being altered. To-day, the compulsory features of public health administration are being overshadowed by the service features. Laboratory diagnostications, the preparation and distribution of biological products, child welfare, infant welfare, maternal welfare, and the like are in full swing and are receiving increasing appropriations. It should be remembered that these matters are supported under the taxing power and that appropriations are justified only in so far as they are for the public benefit and as far as the results warrant the cost. Public economy, not altruistic idealism, should control governmental service. Altruism blossoms as a flower in individual lives, but dispersed by government, its growth is rank and demoralizing. Governmental attempts to benefit individual health and welfare are apt to be inordinately expensive.

At present, some of the ablest health officers are seeking to shift the burden of minor matters of sanitation, often spoken of as "nuisances", to the police departments, to turn the collection and disposal of refuse over to the department of public works, to allow the building department to administer the plumbing laws, to place smoke control under some other agency. In short, we are facing a possible administrative cleavage between the "new public health", as it is sometimes called, and sanitation.

### Administrative Relationship

In health departments, as in other branches of government, there is a tendency toward the single executive type of organization. This may be merely a natural reaction from too much democracy, but it is also an indication that the public service activities in the interest of health are overshadowing the quasi-legislative and quasi-judicial functions of the older board of health. It is natural and proper that the executive of a single-headed health department should be a doctor of medicine; it is also natural that a doctor of medicine should be more interested in and more competent to handle matters of health on their personal side than to deal with matters of sanitary engineering. If the single executive type of organization is to remain, it may be wisest to separate sanitation from the health department and, perhaps, couple it with safety, uniting the various environmental factors into a new department of government in charge of a sanitary engineer.

A serious objection to this separation would be that the people, their political representatives, and even the Courts, would gradually come to hold the idea that sanitation is distinct from health; and this would lead to a loose application of the police power in matters of sanitation and to a possible lowering of the sanitary morale of the community. On the other hand, it is not right to use the word, health, as a club to extort appropriations and secure needlessly harsh regulations, or to bolster up systems of license for plumbers or engineers.

The speaker does not favor this administrative separation, at least as far as State departments are concerned. He cannot forget the phenomenal success of the Massachusetts State Board of Health in its early days, when for a quarter of a century two men dominated its activities—one a physician, the other an engineer, Dr. Henry P. Walcott and the late Hiram F. Mills, Hon.

M. Am. Soc. C. E. It seems that the proper balance between the personal and environmental factors relating to public health can best be secured if the State organization is that of a board of health, the membership of which includes not only physicians, but an engineer, a lawyer, a business man, a manufacturer, and a farmer. At present, the service activities appear to be paramount in interest and receive the largest appropriations, but any one who has served in a State department of health knows that the quasi-legislative and quasi-judicial functions are exceedingly important. In the speaker's opinion, a board composed chiefly of laymen can legislate better than an individual or a group of specialists. Perhaps, a satisfactory arrangement would be to have a State board of health with two principal departments, one of epidemiology and hygiene, and one of sanitation; the first headed by a physician trained in public health, the other headed by a sanitary engineer also trained in public health; the first taking care of health on its personal side, the other dealing with environmental problems in the interest of health. A subdivision of a health department which makes sanitary engineering merely one of half a dozen divisions unduly minimizes the importance of the environment as a health factor, and leads to an illogical budget and an inferior engineering personnel.

In cities, the sanitary activities, if removed from the local board of health, may have to be scattered somewhat in the interest of economical operation and public convenience, yet as far as possible they should be kept together. Cities differ in size and a distribution of functions appropriate to a city of 50 000 people might be quite inappropriate to a city of 500 000. There might be economy, for example, in placing refuse collection, street cleaning, paving, etc., in a street department or a department of public works. It would be a convenience to prospective builders if all building permits, covering construction matters, plumbing, etc., could be concentrated under one head.

The main thing to keep in mind is the principle that health is dependent on two factors: Man and his environment. The pendulum must not be allowed to swing too far in either direction. The speaker is certain that an engineer is well within his prerogative when he counsels equilibrium in the various activities which have to do with health and life.

## Present Status of Sanitary Engineering

The present status of sanitary engineering is in some respects not what it should be. In the U. S. Army during the World War, matters of sanitary engineering were divided between three departments: The Engineer, the Quartermaster, and the Medical. In the Medical Department, the Sanitary Corps was a catch-all and not a well-organized branch of the service. Its officers did not have rank and grade commensurate with their service or an equality with the medical officers. In the U. S. Public Health Service, the engineers do not now have the same status as the medical men. It is time that leaders in education, in public health, and in municipal government gave serious consideration to the re-organization of the Sanitary Service. It is time that the Federal Government took steps to re-organize the sanitary work of the U. S. Public Health Service and the Army and the Navy. This can be done better now, in times of peace, than at some future time when an emergency arises.

A personnel bill to promote the efficiency of the Public Health Service, which among other things authorizes the commissioning of sanitary engineers and gives them rank, grade, and pay equivalent to medical officers, has been prepared by the U. S. Treasury Department and presented to Congress for consideration. Engineers should stand solidly behind this measure as a first step in promoting the status of sanitary engineering in the U. S. Public Health Service. With engineers regularly commissioned, it will be easier to organize them into a separate section or division of the Service, a second step which should soon follow the first. At present, there are only about twenty-five sanitary engineers in this Service; there is no sanitary engineering branch of the Hygienic Laboratory; no engineer on its Advisory Board. Yet, there are far-reaching sanitary problems which should be receiving attention at Washington, problems of the pollution of interstate streams and coastal waters, problems of water purification and the protection of ground-water supplies, problems of refuse disposal, air pollution, and the like. The speaker does not mean an extension of Federal control over these matters—believing that State administration, State co-operation and organization of river districts offer better solutions—but scientific studies carried on in a way that can be done only by the Federal Government, with ample means and an able personnel. The U. S. Public Health Service is now not well balanced as between medical officers and engineers.

Another example of unbalance may be seen in the American Public Health Association, the Association which, par excellence, represents public health in this country. Although it has an active Section of Sanitary Engineering, there is now only one practicing sanitary engineer among its forty-five elected Counsellors; in 1923, there was none. In the programs of the general sessions of the annual meetings, there have been practically no papers or addresses dealing with problems of environment. The blame for this situation lies with the engineers as well as with the medical officers. Sanitary engineers fail to take an interest in the personal side of health protection, and medical health officers are too much engrossed in that side to give deserved attention to the environmental side.

The International Health Board of the Rockefeller Foundation has no engineer member. The Health Committee and the Health Section of the Secretariat of the League of Nations have no engineer members. Sanitary engineering in the State departments has been better recognized, and the good results are evident. In cities, there is much to be desired, but before local administrative plans are determined, the main question should be given thorough discussion: Shall public sanitation be separated administratively from public health, or shall there be co-ordination on an equal footing?

The number of students electing sanitary engineering in engineering schools is at a low ebb. It is now difficult to obtain enough well educated young engineers. Unless more students enter this field, there will soon be a serious shortage. The speaker will not attempt to explain why students are looking toward business and industry rather than public service, or why they are seeking positions where their work is paid for by fees rather than by salary. These are general tendencies not confined to sanitary engineering. It is

probable that, in the past, sanitary engineering programs were poorly constructed, that the attempt to educate sanitary engineers by combining engineering, chemistry, and biology in substantially equal parts was a mistake. A sounder method is to develop the civil engineer first and then lead him into the special field of sanitary engineering.

The status of sanitary engineering has been referred to, not because sanitary engineers are dissatisfied with their lot—few of them are—but because the public health movement of to-day needs the steadying influence of the engineer, needs his critical, logical, calculative mind, his powers of construction and organization, and his ability to determine costs and relative values.

## Life

Health is not life, but a state, quality, or condition of life. Sanitation, hygiene, and all other health agencies are but means to an end. Their chief purpose should be to develop richer and fuller individual lives and a higher civilization. To save life is not enough. What is the gain in saving the lives of children in the slums, if they whose lives are saved must always live in slums? Human happiness and usefulness, not mere longevity, should be the goal. It may be argued with much force that the health promotive agencies which act positively and contribute to fullness of life are really more important than those protective agencies which act negatively and merely prolong life without advancing it to a higher plane. Let it be remembered that throughout a man's life until old age sets in the chance of living is greater than that of dying. The quality of life is of more account than the event of death.

This is said to be a material and pleasure-seeking age. At the same time, it is called the age of the engineer, the age of power. If these statements are true, a great responsibility rests on the engineer. There is a strong tendency for engineers to lose sight of the great elements of life. The time-honored professions of theology, law, and medicine are fundamentally personal in the service they render; the Engineering Profession is not only newer than the others, but deals chiefly with materials and natural forces, and its benefits are collective rather than individual. Engineering will be recognized as a profession only as engineers give personal disinterested service and as they make people realize the close relation of their work to life, liberty, and the pursuit of happiness.

Looking back a century or more, the beginnings of industrialism, the rise of the factory, and the growth of cities can be seen as a result of science and engineering; one can see the early evils of industrialism leading to a humanitarian movement, the great sanitary awakening at the beginning of the Victorian era. Then came Pasteur, the science of bacteriology, and the new public health. Now, with advanced ideas of industrial humanics, housing, and city planning, we are entering on a new era of sanitation, and once more the engineer must lead. Industrial revolution and a rude economic awakening are already upon us. The age of power is becoming the age of super-power; city planning has already become regional planning; decentralization of population is coming. The new problem is not so much how to educate a few sanitary

engineers, but how to educate all engineers to work in the interest of health and life.

The sanitary engineer especially comes close to life. A knowledge of biology is fundamental to his success. His work is a contribution to the great elements of life. He has opportunities to protect and promote health and comfort. He has opportunities to protect the beauty of trees, streams, lakes, and ocean shores, to build beautiful structures, to utilize the latent beauty of falling water. Although burying much of his work under ground, he makes possible the development of noble city plans. Through organized cleanliness, he makes it possible for the beauties of architecture to be revealed. The new architectural roof motive, already evident in New York, resulting from the restriction of building heights is an outgrowth of a sanitary demand for adequate light and air. Through insistence on individual responsibilities in public sanitation there is being developed a moral sense of duty and a spirit of co-operation, which is the very soul of civilization.

It is well, at times, for sanitary engineers to stop and look at these things, and even to indulge in a little self-conscious professional pride, in order to encourage young men to enter this field of work. Sanitary engineering is a worthy, beneficent calling; it offers a sufficient living, opportunities for scientific achievement in varied fields, and many pleasures. Those who began work at the time the spirit of Pasteur was stimulating all public health efforts, have witnessed striking changes in living conditions, due to improved sanitation of air, soil, water, food, streets, houses, factories, and public buildings. They have seen pestilences yield to science; the art of water purification accelerate until a grossly polluted public water supply is now almost a thing of the past; and the art of sewage treatment push toward the goal, but with many yards yet to go. They have seen city and regional planning develop as the art which is to save cities from their own colossal folly. They have seen chemistry, biology, and engineering unite in solving sanitary problems. They are now seeing young men from all parts of the world coming to America to learn the arts of sanitation and health promotion and returning to their several countries to practice them and teach them in their own universities. World-wide sanitation is looming ahead as a Twentieth Century possibility, for an age is fast approaching in which the unit of environment is the world.

# REACTIONS FOR A PARTICULAR TYPE OF UNSYMMETRICAL ARCH

## Discussion*

By Carl B. Andrews, Assoc. M. Am. Soc. C. E.†

Carl B. Andrews,‡ Assoc. M. Am. Soc. C. E. (by letter).§—In the discussions of this paper, it is pointed out that the moments occurring throughout the unsymmetrical arch are the same as those which occur in the symmetrical arch, provided that the loading is the same in each case.

If the designer is platting influence lines for the moments at different sections, knowledge of this fact may save him some work. The total moments in two such structures, however, will not generally be the same at corresponding points, because of differences in the dead loads.

The subject-matter of the paper will be valuable mainly to those who use the method of coefficients for the statically indeterminate quantities, $H_1$, $V_1$, and $M_1$. These coefficients are the values of these respective quantities which are obtained when a unit load is placed on the arch at any desired point. Having the coefficients, the actual quantities for any loadings are easily obtained.

It has been shown‖ that the axis of one symmetrical arch may be projected by any number of parallel projections into the axis of another symmetrical arch, and that, provided the ring-thickness functions, $\Delta$, in the new arch bear a constant ratio to the corresponding values in the original arch, the coefficients, $H_1$, $M_1$, and $V_1$, change, as follows: $M_1$ and $M_2$ change directly as the span; $H_1$ changes directly as the span and inversely as the rise; and $V_1$ does not change at all.

If a drawing of the arch axis, with force and equilibrium polygons, is thus changed by any number of parallel projections, the new force and equilibrium polygons will be correct for the new symmetrical arch. The scales of the drawings may change in the projection. Similarly, an influence line for one arch may be projected into another influence line for another arch.

Professor Rathbun's¶ general projection of a symmetrical axis into an unsymmetrical one is an abbreviation of the above work. Evidently, the result of his single projection by oblique rays on any plane may be obtained by successive projections from an original symmetrical form into other symmetrical forms, and, finally, into an unsymmetrical one, and the result will be as he states.

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* Discussion of the paper by Carl B. Andrews, Assoc. M. Am. Soc. C. E., continued from December, 1923, *Proceedings*.

† Author's closure.

‡ Prof. of Eng., Univ. of Hawaii, Honolulu, Hawaii.

§ Received by the Secretary, February 16, 1924.

‖ "Arch Coefficients," by M. A. Howe, M. Am. Soc. C. E., *Engineering News*, June 2, 1910.

¶ *Proceedings*, Am. Soc. C. E., December, 1923, p. 2086.

The use of arch coefficients, when thus extended by the process of parallel projection from an original form into other symmetrical or unsymmetrical forms, may result in a great saving of labor in the design of any particular arch, provided the coefficients have already been determined for a basic arch of a form which will project into a suitable arch for the case in hand. An engineer specializing in arch design should be able, by the use of coefficients for a variety of basic arches, to lighten his computation work materially.

# PERIODIC FLUCTUATIONS OF RAINFALL IN HAWAII

## Discussion*

By Joel B. Cox, Assoc. M. Am. Soc. C. E.†

Joel B. Cox,‡ Assoc. M. Am. Soc. C. E. (by letter).§—The studies presented in the paper were made in the latter part of 1921, and Table 12‖ shows the application of the cyclic theory made at that time, giving the expected monthly precipitation of the East Maui water-shed for 1922 and 1923. It is now possible to compare this estimate with the observed rainfall, which has been done in Table 14. The improvement due to the cyclic estimate during this 2-year period is 7.8% of the standard deviation of the similar estimate without the cyclic theory, whereas the average error is reduced 5.66 per cent. The total discrepancy for the 2 years is 20.20 in. by the cyclic theory and 45.14 in. by the ordinary method; an improvement of 55 per cent. Such an application to future records is, of course, the test that will determine the usefulness of the cyclic theory, although considerable time must elapse before more definite conclusions can be drawn from Hawaiian records.

Mr. Grunsky states¶ that the available observations from many parts of the world seem to show that a cycle equal to one-ninth of the sun-spot period is a real one, and not attributable to accidental causes. If there is a real cycle of one-ninth the sun-spot period, a cycle of one-third the sun-spot period, including three of the smaller cycles, is also real. In Fig. 6,** the two curves marked, "16-Year Record—6 East Maui Stations" and "16-Year Record—147 Stations", both suggest the existence of the smaller cycle within the 3.7-year period.

It is to be noted that in one carefully made analysis of the period of one-ninth the sun-pot period,‡‡ it was found that the length fluctuated considerably in synchronism with that of the main solar cycle. This effect is also observable in the Hawaiian records, although as it does not now seem possible to predict the duration of a sun-spot period in advance, it is not possible as yet to use this variation in the preparation of estimates. A similar difficulty was found by Mr. Alfred J. Henry in studying temperature variations.§§ In the Synopsis of his article, it is stated that:

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* Discussion of the paper by Joel B. Cox, Assoc. M. Am. Soc. C. E., continued from March, 1924, *Proceedings*.

† Author's closure.

‡ Civ. Engr., McBryde Sugar Co., Ltd., Eleele, Kauai, Hawaii.

§ Received by the Secretary, February 27, 1924.

‖ *Proceedings*, Am. Soc. C. E., October, 1923, p. 1719.

¶ *Loc. cit.*, December, 1923, p. 2089.

** *Loc. cit.*, October, 1923, p. 1707.

‡‡ "A Possible Rainfall Period Equal to One-Ninth the Sun-spot Cycle", by Dinsmore Alter, *Monthly Weather Review*, February, 1921.

§§ "Temperature Variations in the United States and Elsewhere", by Alfred J. Henry, *Monthly Weather Review*, February, 1921.

"Both tropical and temperate zone stations show very clearly the persistence of short period variations of about forty months period. One of the chief characteristics of the data is the tendency of any marked variation in the temperature to be followed by another one of opposite phase almost immediately. While this tendency amounts to almost certainty, it is useless for forecast purposes because there is no means of discovering the precise duration of any existing phase."

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TABLE 14.—COMPARISON OF OBSERVED AND PREDICTED RAINFALL, EAST MAUI WATER-SHED.

| Month. | Expected rainfall, ordinary estimate. | Expected rainfall, cyclic theory. | Actual rainfall. | (Column 4 — Column 2). | (Column 4 — Column 2)². | (Column 4 — Column 3). | (Column 4 — Column 3). |
|---|---|---|---|---|---|---|---|
| (1) | (2) | (3) | (4) | (5) | (6) | (7) | (8) |
| **1922.** | | | | | | | |
| Jan | 11.55 | 19.2 | 45.96 | 34.41 | 1 184.05 | 26.76 | 716.10 |
| Feb | 17.59 | 27.1 | 22.61 | 5.02 | 25.20 | — 4.49 | 20.16 |
| Mar | 18.41 | 24.9 | 29.18 | 10.77 | 115.99 | 4.28 | 18.32 |
| Apr | 20.25 | 24.0 | 15.79 | — 4.46 | 19.89 | — 8.21 | 67.40 |
| May | 15.65 | 17.4 | 10.05 | — 5.60 | 31.36 | — 7.35 | 54.02 |
| June | 12.84 | 14.0 | 3.32 | — 9.52 | 90.63 | —10.68 | 114.06 |
| July | 15.52 | 16.8 | 7.19 | — 8.33 | 69.39 | — 9.61 | 92.35 |
| Aug | 17.78 | 18.7 | 13.20 | — 4.58 | 20.98 | — 5.50 | 30.25 |
| Sept | 13.30 | 13.5 | 13.78 | 0.48 | 0.23 | 0.28 | 0.08 |
| Oct | 14.36 | 14.5 | 18.13 | 3.77 | 14.21 | 3.63 | 13.18 |
| Nov | 18.66 | 17.8 | 22.24 | 3.58 | 16.40 | 4.44 | 19.71 |
| Dec | 18.82 | 16.7 | 4.55 | —14.27 | 203.63 | —12.15 | 147.62 |
| **1923.** | | | | | | | |
| Jan | 11.55 | 10.3 | 45.61 | 34.06 | 1 160.08 | 35.31 | 1 246.80 |
| Feb | 17.59 | 17.0 | 15.13 | — 2.46 | 6.05 | — 1.87 | 3.50 |
| Mar | 18.41 | 19.5 | 13.09 | — 5.32 | 28.30 | — 6.41 | 41.09 |
| Apr | 20.25 | 22.3 | 24.21 | 3.96 | 15.68 | 1.91 | 3.65 |
| May | 15.65 | 16.4 | 6.45 | — 9.20 | 84.64 | — 9.95 | 99.00 |
| June | 12.84 | 11.4 | 8.63 | — 4.21 | 17.72 | — 2.77 | 7.67 |
| July | 15.52 | 11.8 | 13.45 | — 2.07 | 4.28 | 1.65 | 2.72 |
| Aug | 17.78 | 13.3 | 9.65 | — 8.13 | 66.10 | — 3.65 | 13.32 |
| Sept | 13.30 | 11.2 | 14.47 | 1.17 | 1.37 | 3.27 | 10.69 |
| Oct | 14.36 | 14.1 | 19.96 | 5.60 | 31.36 | 5.86 | 34.34 |
| Nov | 18.66 | 20.8 | 16.26 | — 2.40 | 5.76 | — 4.54 | 20.61 |
| Dec | 18.82 | 21.7 | 41.69 | 22.87 | 523.04 | 19.99 | 399.60 |
| Total | 389.46 | 414.40 | 434.60 | 45.14 | 3 736.34 | 20.20 | 3 176.20 |
| Average deviation | ...... | ...... | ...... | 8.593 | ......... | 8.107 | ......... |
| Standard deviation | ...... | ...... | ...... | ...... | 12.477 | ...... | 11.504 |

Mr. Grunsky states that the Hawaiian data are inadequate to make the demonstration conclusive. The extent to which this is true is shown in Table 9 and its discussion.* As pointed out there, no matter how true and real may be the existence of a periodic tendency suspected, a strict and conclusive statistical proof is impossible until a sufficiently long record is obtainable. For Hawaii alone, this will be impossible for many years. It is to be hoped, however, that the co-ordination of studies from different parts of the world, especially where long-term tropical records are available, will hasten this time.

Mr. Hall† asks how the 3.7-year period was chosen for study. The steps were about as follows: First, it was noticed that the rainfall of a single station—Kailua, Maui—showed a definite tendency toward a 3 or 4-year period.

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* *Proceedings*, Am. Soc. C. E., October, 1923, p. 1714.

† *Loc. cit.*, March, 1924, p. 343.

This was discovered in plotting the record for use in run-off estimates. The literature available suggested 11.1 years, 3.7 years, and 15 months, as possible theoretical periods. As the 3.7-year period was of about the length sought, the attempt was made to fit it to the extended Hawaiian record of Table 2,* with the results as published.

Referring again to Mr. Hall's comments, it is true, of course, that accidental effects are far larger than any systematic variation discoverable. In the Hawaiian Islands, the effect of the undisputed and very real annual cycle is so heavily overlaid by accidental variations that a considerable length of record is needed to determine its true effect. This obscuring of systematic tendencies by the great accidental variations is inherent in the nature of the problem. It is a misconception to suppose that the claim was put forward to the discovery of a cyclic correction which would eliminate a large proportion of the unknown causes of variation. If such a cycle existed, it would undoubtedly have been discovered long ago, and its demonstration would be easy and the proof self-evident. The intent of Fig. 9† and its discussion was exactly to eliminate such an impression. With all that is known of the causes and cyclic tendencies of Hawaiian rainfall, an estimate of the rainfall for a given future month has a probable error of about 35 per cent. If the cyclic theory can reduce this probable error from 40% to 35%, as indicated by the records thus far studied, the writer believes that a significant result has been obtained.

Mr. Hall raises the question of the economic value of an improvement in the knowledge of Hawaiian rainfall by such an amount. The writer believes that every such small improvement in the knowledge well repays the effort involved. The Hawaiian sugar industry has an annual value of about $60 000 000 derived from a cropped area of 115 000 acres. A large part of this business consists of a gamble in regard to the rainfall. For an average plantation, with the usual rainfall variation, a 5% error in determining the most economical area to cultivate will entail an average loss of $2 per acre harvested. This would result in an annual saving of $230 000 for the Islands. The studies indicate that a cyclic influence of this average magnitude may be both real and predictable.

In conclusion, it seems apparent that a rigid statistical proof of the reality of such systematic variations in precipitation as have been studied must await the accumulation of longer homogeneous records than are now available, but that experience with the Hawaiian records indicates that similar studies by engineers and meteorologists dealing with tropical problems will not only hasten the day when certainty may be attained, but, in the meantime, the use of such data has a fair chance of giving greater accuracy and precision to rainfall estimates and thus increase their value to the industry for which they are prepared.

* *Proceedings*, Am. Soc. C. E., October, 1923, p. 1701.

† *Loc. cit.*, p. 1717.

# COMPARATIVE TESTS ON EXPERIMENTAL DRAFT-TUBES

## Discussion*

BY MESSRS. WILLIAM J. RHEINGANS, LEWIS F. MOODY, H. BIRCHARD TAYLOR, J. F. ROBERTS, D. J. MCCORMACK, and E. S. LINDLEY.

WILLIAM J. RHEINGANS,† JUN. AM. SOC. C. E.—As one of the four men of the testing crew who assisted in making this series of tests, the speaker presents this discussion especially in behalf of the Hydraucone and its designer, Mr. W. M. White.

During the past four years, hydraulic engineers have devoted more time and effort to draft-tube design than ever before and justly so, due to the ever-increasing demand for higher efficiency and to the fact that draft-tube design has not kept pace with the design of runners. This phase of turbine work received its impetus in the fall of 1917 when the Allis-Chalmers Manufacturing Company submitted the White Hydraucone, with other forms of draft-tubes, to the then Hydraulic Power Company at Niagara Falls for comparative tests. On the publication by Mr. White of a paper‡ in 1921, the subject of improvements in draft-tube design was brought before the Engineering Profession. Soon tubes of various shapes were designed, theories formed and presented to the public, and experiments performed, so that this might be designated as the "Draft-Tube Age" in hydraulic turbine design.

Much has been written about the advantages and efficiencies of the various forms of draft-tubes. Some engineers based their claims on tests in private hydraulic laboratories, others on the results in power plants, but the comparisons were always tainted with commercial considerations. The Alabama Power Company is, therefore, the first to have a series of tests made on a large scale and under identical conditions by disinterested parties using different designs of draft-tubes.

Because of the high efficiencies obtained with the White Hydraucone and because of its radical departure from the shape of the curved draft-tube, it is natural that the later designs should use its fundamental principles.

Just what is meant by "hydraucone"? The following is from the Hydraucone Patent (No. 1 223 843), filed by Mr. White, in 1915:

"By the term 'hydraucone action' of water I mean that action of the water which occurs as the stream impinges against the surface and is deflected therelong". And, again, "In the drawings I have shown the hydraucone chamber with flat bottom for an impinging surface, but such impinging surface may be conical or convex or concave. The shape of the free hydraucone may be

* Discussion of the paper by C. M. Allen, M. Am. Soc. C. E., and I. A. Winter, Esq., continued from March, 1924, *Proceedings*.

† Milwaukee, Wis.

‡ "The Hydraucone Regainer, Its Development and Applications in Hydro-Electric Plants", *Transactions*, Am. Soc. Mech. Engrs., Vol. 43 (1921), p. 255.

different depending upon the peculiar form of base used. I make the walls of the chamber to a shape which provides an enclosed conoidal chamber of slightly increasingly greater capacity in the direction of flow than that required to conform to the shape of the free hydraucone which would tend to form on impact with the particular form of base used."

What could be simplier than this? It is desired to regain the energy in a stream of water by reducing its velocity and, at the same time, changing its direction. Let a stream of water impinge against a surface, which, according to the quotation from the Hydraucone Patent, is not limited to a flat surface but may also be conical, concave, or convex. The shape of this stream as it impinges against any given surface is determined by any of several methods and around it is built a conoidal chamber of "slightly increasingly greater" capacity in the direction of flow than that required to conform to the shape described. This will change the velocity of the water into pressure and, at the same time, change its direction with as little loss as possible.

The term, hydraucone, is used in its broader sense as defined in the patent; in its physical aspects, this form is characterized by the radial flow from a common center line and by the narrowness of the throat vertically. Both these features are common to all the leading tubes described by the authors.

The significant point of this whole series of tests is its complete justification of the hydraucone principle. As a matter of fact, the profile of the best hydraucone, the best spreading draft-tube, and Wellman-Seaver-Morgan tube are almost identical; together they compose a distinct class radically different from the common curved tube.

In Fig. 31 is shown the hydraucone (Type B) superimposed on the Wellman-Seaver-Morgan tube (Type H). Two of the important fundamental features of the hydraucone are the shape of the conoidal chamber (the part, *A-B*) and the distance, *B-C*, sometimes called the throat opening. These features correspond closely in the two tubes. Starting where the water discharges from the runner, the hydraucone is shaped so that the area is increased until, at the central part of the curve, *A-B*, the area begins to decrease until the throat opening has been reached, after which it increases. This feature is shown in Fig. 32, in which the areas at different sections are plotted as ordinates.

Fig. 33 shows the areas of the Wellman-Seaver-Morgan tube plotted on a similar basis. The forms of the curves from the two draft-tubes are nearly identical.

It cannot be emphasized too strongly that a single series of tests of this kind should not be taken as applying to hydraulic turbine settings in general, but only to the particular runner and the particular setting under investigation, as pointed out by the authors. Unfortunately, as far as general conclusions are concerned, the runner used has the smallest whirl component in its discharge and the most uniform velocity across its discharge of any with which the speaker is familiar. It is due to this fact that all the tubes appeared to such good advantage, even some of the older curved types. It is only in the light of a large number of similar test series made with runners differing in type, discharge, characteristics, and size, that general conclusions may be reached. The development of the hydraucone was the result of such a com-

prehensive test program. This is mentioned on account of the danger of drawing broad conclusions from an isolated series of tests. Keeping these

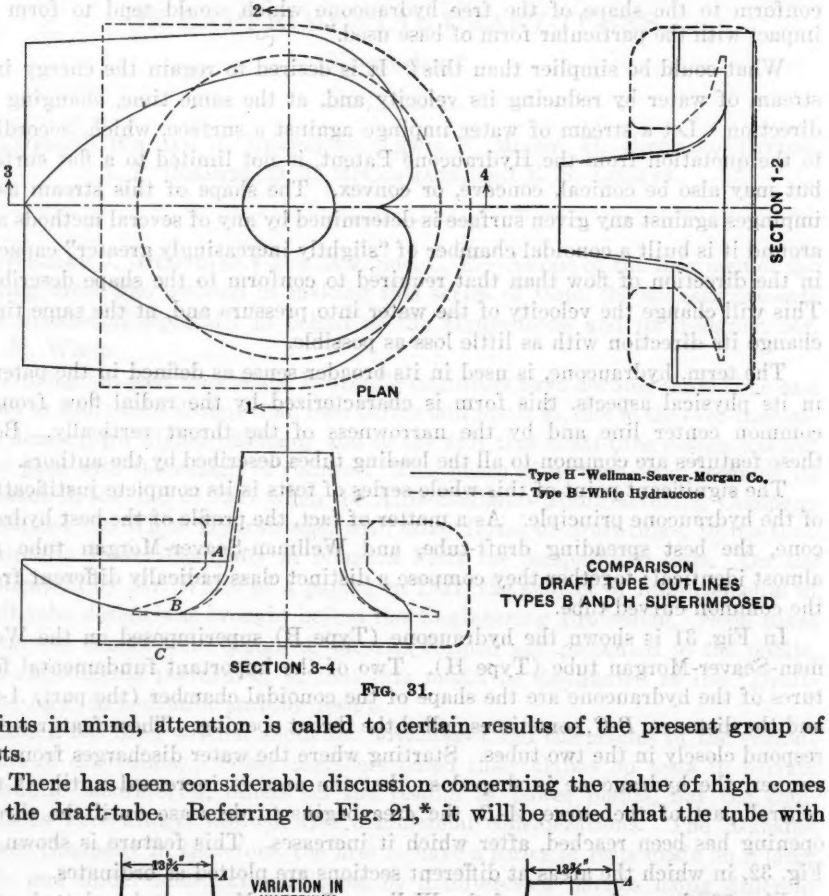


FIG. 31.

points in mind, attention is called to certain results of the present group of tests.

There has been considerable discussion concerning the value of high cones in the draft-tube. Referring to Fig. 21,* it will be noted that the tube with

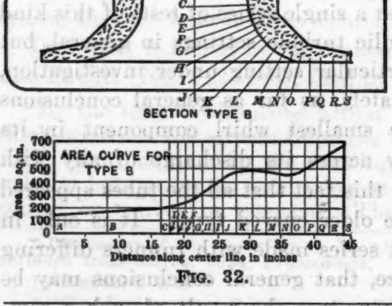


FIG. 32.

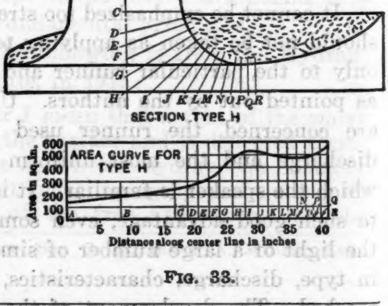


FIG. 33.

* *Proceedings*, Am. Soc. C. E., November, 1923, p. 1834.

a high cone gives a maximum efficiency only 0.2% higher than one with a flat plate, assuming the correct curve for the latter as that drawn between the circles and triangles representing the two tests. The maximum horse-power is the same in both cases, the only important difference being 1% increase in efficiency at half load for the high cone.

Thus, it is seen that the maximum efficiency is only 0.2% lower, while the maximum horse-power is actually greater without a cone than with the high cone for which the tube was especially designed. As far as efficiency and horse-power are concerned, the high cone in this type of draft-tube, and for a runner with a specific speed of 70, is of no special benefit. This is possibly explained by the small amount of rotation in the discharge. These cones are found to be of advantage in certain instances, but for runners of medium specific speed, they have no appreciable advantage and, consequently, their form is not always utilized for commercial installations. Runners of certain specific speeds, notably the higher ones, operate more advantageously with a conical center in the hydraucone. For this reason, cones were used in some hydraucone installations as early as 1916.

Another interesting result is the increase in efficiency with certain changes in the Moody spreading draft-tube (compare Fig. 12* with Fig. 23†). The diameter of the throat was increased from 3⅞ in. to 4⅞ in. and the diameter of the bell from 3 ft. 7½ in. to 3 ft. 10½ in. The corresponding dimensions of the hydraucone are 4 19/32 in. and 3 ft. 11½ in. Also, the shape of the conoidal chamber of the Moody tube was changed to conform more closely to the shape of the hydraucone. This shows that the efficiency of the Moody tube increases as it conforms more closely to the dimensions of the hydraucone.

Incidentally, the numerous changes—smoothing out curves, etc.—made in all the other tubes, were not applied to the hydraucone, the original design being used. This is evidence of the intimate knowledge of the hydraucone action of water on the part of the engineers of the Allis-Chalmers Manufacturing Company responsible for its design. If it were necessary to build a hydraucone for any other set of conditions, similar satisfactory results would result without experimenting. The work leading up to the present perfection of the hydraucone has required more than 10 years. The confidence expressed in it by hydraulic engineers is evidenced by the fact that using this principle, the Allis-Chalmers Manufacturing Company to date has installed 1 020 000 h.p., or one-tenth of the developed horse-power in the United States. Most noteworthy among the installations are the 70 000-h.p. unit at Niagara Falls, which is the largest single unit in the world, and the twelve 45 000-h.p. units for the Quebec development in Canada, comprising a total of 540 000 h.p., the largest single turbine contract ever awarded.

From a study of the results obtained with curved draft-tubes, it will be noticed that Type E is prominent. This design is based on the flattened elbow used by Gardner S. Williams, M. Am. Soc. C. E., as early as 1911, and due credit should be given to him as a pioneer in developing this form of elbow tube.

* *Proceedings*, Am. Soc. C. E., November, 1923, p. 1830.

† *Loc. cit.*, p. 1835.

Type M, which is a modification of Type E, does not give a fair comparison with the other tubes in this series of tests, since it is 3 in. higher; this changes the conditions of the test, because all the draft-tubes were built to conform to one height. It is well recognized that, within certain limits, the greater the distance from the discharge of the runner to the floor of the tail-race, the more efficient the draft-tube which can be built in that space.

As the speaker was a witness to these tests from beginning to end, he can certify to the strictness and excellence of Professor Allen's supervision and to the care, precision, and correctness which obtained in making the various readings, computations, and diagrams. Wherever there was the slightest question as to the accuracy of a test, or the setting of the apparatus, an immediate check run was made, and all conditions which might have caused an error were inspected and checked. The main consideration was the accuracy of the results and not the speed with which tests could be made.

There is one feature of the tests which might have affected some of the results obtained. All the draft-tubes were of wood, which when dried out after immersion for some time, had a tendency to warp. As there was quite a lapse of time between the first and second series of tests, this warping was especially noticeable on draft-tubes used in both series. Some of the draft-tubes, for example, Types H and J, were especially constructed to withstand the warping effects, using thick walls of hard wood and painting them with a water-resisting varnish. Whenever it was found that one had warped badly, it was forced back into shape.

From the manufacturer's standpoint, appreciation is expressed for the opportunity to discuss a series of tests of this nature, to Professor Allen for his untiring efforts in overcoming all the obstacles of the work, and to the Alabama Power Company for disregarding costs in favor of actual facts.

LEWIS F. MOODY*, ESQ.—In preparing and presenting this paper, the authors have made available many useful data. It would have added still more to the value of the paper if the series of supplementary tests carried out by Professor Allen had been included. These supplementary tests are really necessary to a complete understanding of the turbine performance secured with the draft-tube model, designated as Type K, as it will be noted that the tests made with this tube, as presented in Fig. 23†, are limited in scope, covering only one position of the central core or cone and not including any tests with the form of core shown by dotted lines in Fig. 21‡, that is, where this central core is carried completely through the tube, beginning immediately at the runner. The results of these additional tests are shown in Fig. 34, together with the curves for Tubes H and K taken from Fig. 26§, and a revised curve for the same setting of Tube K made after correcting the adjustments of instruments. The highest curve shows the results attained with Tube K equipped with its high central cone.

* Cons. Engr., I. P. Morris Dept., Wm. Cramp & Sons Ship & Engine Bldg. Co., Philadelphia, Pa.

† *Proceedings*, Am. Soc. C. E., November, 1923, p. 1835.

‡ *Loc. cit.*, p. 1834.

§ Tube K is incorrectly marked *A* in that diagram.

For a number of years, the speaker has been advocating the use of a form of draft-tube generally known as the "Spreading" tube, in which the velocity deceleration is effected in an annular passage contained between concentric surfaces of revolution flaring from an axial toward a radial direction—a form of tube which has been applied to more than 100 units of nearly 2 000 000 total horse-power. This discussion refers to the performance of this form of tube as compared with that of others. Any impression that the best results were given by a form of tube other than the spreading draft-tube, can be shown from a study of the paper to be unfounded. This fact would have been made still clearer, if the supplementary tests (Fig. 34) had been included

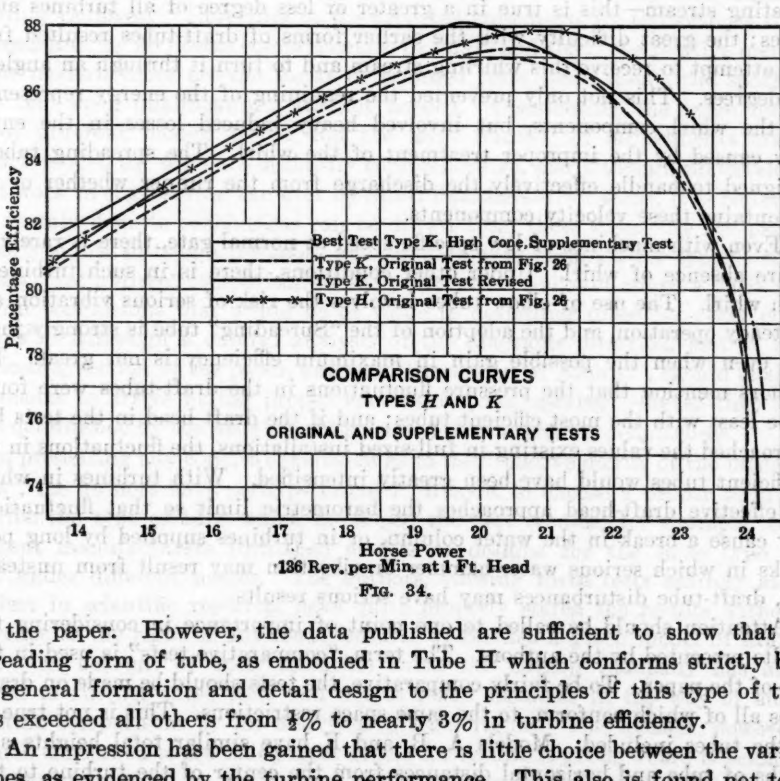


Fig. 34.

in the paper. However, the data published are sufficient to show that the spreading form of tube, as embodied in Tube H which conforms strictly both in general formation and detail design to the principles of this type of tube, has exceeded all others from ½% to nearly 3% in turbine efficiency.

An impression has been gained that there is little choice between the various tubes, as evidenced by the turbine performance. This also is found not to be verified on close study.

The speaker believes that few engineers consider even one-half of 1% difference in turbine efficiency unimportant; in addition to this aspect of the question, it should be considered that the turbine used was not of high specific speed, and that a small difference in turbine efficiency represents a material difference in the performance of the draft-tube itself. Thus, when the turbine is operated with Tube K equipped with a high central cone, at its point of best efficiency, the velocity head of the water discharged from the runner is between 6½ and 7% of the total head on the turbine. Therefore, the greatest effect in turbine efficiency that a draft-tube could produce in the performance

of this particular turbine, if its efficiency varied from zero to 100%, would be less than 7%; a difference in turbine efficiency of 1%, therefore, would imply a difference of 15% in draft-tube efficiency. As regards the best turbine performance obtained and that next in order of excellence, these involve differences in efficiency of the draft-tubes themselves of from 7 to 15 per cent. In the case of Tubes K and C, the difference is nearly 45 per cent. In turbines of higher speed, these differences would be greatly magnified, due not only to the relatively higher velocity heads, but to the greater whirl or rotational components in the velocity of discharge from the runner.

The water discharged by a runner enters the draft-tube in the form of a rotating stream—this is true in a greater or less degree of all turbines at all gates; the great difficulty with the earlier forms of draft-tubes resulted from the attempt to receive this whirling stream and to turn it through an angle of 90 degrees. This not only prevented the regaining of the energy represented by the whirl components, but involved heavy induced losses in the entire flow caused by the improper treatment of the whirl. The spreading tube is designed to handle effectively the discharge from the runner whether or not it contains these velocity components.

Even with machines of low specific speed at normal gate, there is rarely an entire absence of whirl. Under other conditions, there is in such turbines a high whirl. The use of elbow tubes involves the risk of serious vibration and unsteady operation, and the adoption of the "Spreading" tube is strongly justified even when the possible gain in maximum efficiency is not great. The authors mention that the pressure fluctuations in the draft-tubes were found to be least with the most efficient tubes; and if the draft head in the tests had approached the values existing in full-sized installations, the fluctuations in the inefficient tubes would have been greatly intensified. With turbines in which the effective draft-head approaches the barometric limit so that fluctuations may cause a break in the water column, or in turbines supplied by long penstocks in which serious water-hammer or vibration may result from unsteady flow, draft-tube disturbances may have serious results.

Attention should be called to one point of importance in considering the results presented by the authors. The term, "comparative tests" is used in the title of the paper. To be fairly comparative, the tests should be made on draft-tubes all of which conform to the same space restrictions. This is not true of all the tubes included. Models A, B, and K have similar total heights and widths of tube and horizontal distances from the center of the turbine to the discharge end, but the other models exceed one or more of these dimensions. Thus, the spreading tube *H* involves deeper excavation and much greater horizontal extension than the spreading tube K; and all the elbow and "eccentric" types involve either greater depth or greater horizontal length, or both, than the spreading tube K. The fact that, in spite of these discrepancies, Tube K gives the best efficiency is creditable to this type; and for equal space restrictions its superiority would be still more evident.

The recent tests carried out at the I. P. Morris Laboratory, under the direction of H. L. Cooper and Company, for the Federal Government, in which all the models tested conformed to the same space limitations, and in which a turbine of higher specific speed was used than in the tests described in the

paper, serve to confirm these conclusions, and show a turbine efficiency for the spreading tube amounting to 1% more than for the tube next in order. It is unnecessary to repeat the results, as they have been published. It should be mentioned that Mr. Winter, one of the authors of the paper, represented H. L. Cooper and Company and the Government on these tests. It is gratifying to be able to note the general agreement of the conclusions derived from these later tests with those of the tests described.

H. Birchard Taylor,* Esq.—The tests described confirm a number of conclusions derived from many tests carried out during the past few years in the I. P. Morris Laboratory of the William Cramp and Sons Ship and Engine Building Company. Referring to the repeated statements by the authors that the tests are "by no means comprehensive or final" and that "the relative merits of the modern tubes are not easily determined from these tests", the speaker feels that such qualifications are not needed. Considering the large number of different models tested and the large number of adjustments made on each model, the tests are decidedly comprehensive, and although no development work in engineering can ever be called final in a strict sense, it is believed that certain definite conclusions can be drawn from these tests. As the paper properly states, although the tubes and runners were designed for specific conditions, "yet a direct comparison of the performances should afford a basis for general conclusions that would be of value in designing tubes for other conditions". Although a careful analysis of the diagrams presented and of the concluding section of the paper, entitled "Discussion of Results", should enable the reader to arrive at the proper conclusions, the speaker believes that the facts determined by the tests should be stated with a greater definiteness.

In presenting any concise conclusion as to the "relative merits of the modern tubes", the authors have perhaps found it difficult to choose a proper term to describe the type of tube which developed the best results; for example, two different manufacturers furnished models or designs for the same type of tube, under different names. The authors, viewing these tests entirely as a problem in scientific research, have quite properly stated the results without reference to the right of any manufacturer to furnish a given model of draft-tube. On account of their scientific value the Cramp Company has also desired to further these tests and not to place obstacles in the way of such scientific comparisons by the assertion of legal rights. The speaker, however, is much concerned to see that proper credit for results is given where it belongs and that there shall be no general misconception regarding the identity of the best type of draft-tube. He therefore desires to emphasize the following points:

*First.*—The models designed by the I. P. Morris Department of the Cramp Company and designated in Table 1† as "Moody Spreading Tube", and Type H, constructed by the Wellman-Seaver-Morgan Company and designated "Concentric Tube", are both, in fact, Moody Spreading Draft-Tubes. The design of this tube has previously been published; and the use of such a tube under a

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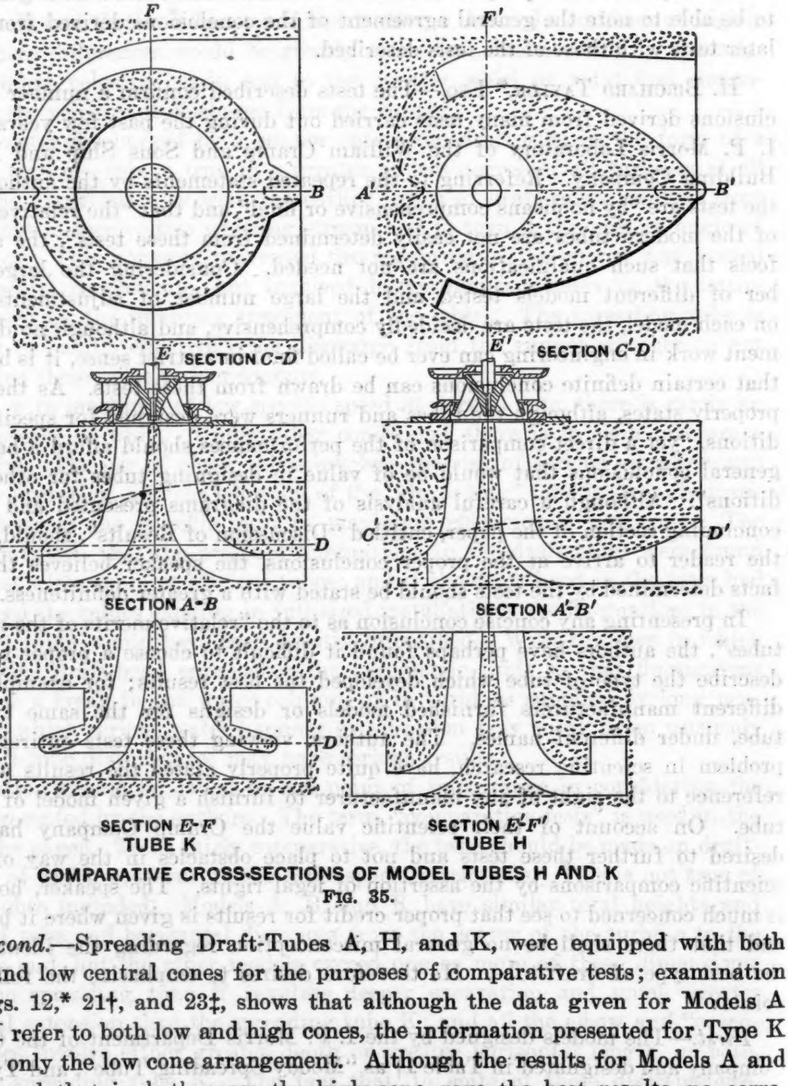
† *Proceedings*, Am. Soc. C. E., November, 1923, p. 1815.

different name should not entitle the company furnishing it to secure credit for its performance. Comparative cross-sections of the model tubes H and K are presented in Fig. 35. It does not require any close analysis to recognize the identity of type of these two models.




COMPARATIVE CROSS-SECTIONS OF MODEL TUBES H AND K

FIG. 35.

*Second.*—Spreading Draft-Tubes A, H, and K, were equipped with both high and low central cones for the purposes of comparative tests; examination of Figs. 12,* 21†, and 23‡, shows that although the data given for Models A and H refer to both low and high cones, the information presented for Type K covers only the low cone arrangement. Although the results for Models A and H showed that in both cases the high cone gave the best results, no corresponding information has been furnished regarding Model K. Moreover, as contrasted with the other tubes, the information for Tube K comprises merely

* *Proceedings*, Am. Soc. C. E., November, 1923, p. 1830.

† *Loc. cit.*, p. 1834.

‡ *Loc. cit.*, p. 1835.

one particular vertical adjustment of the central cone (which, later, was found not to be the best).

Complete information as to the performance of Model K is evidently called for; fortunately, this is available, that is, the test of Model K was actually completed using the high cone furnished with this tube, and the tube was tested with various adjustments of both cones. These results show that a higher efficiency was secured with the low cone when properly adjusted than is shown by the curve presented by the authors, and that the best results of all were secured with the high central cone.

Fig. 36 shows some of the results, those for Type K having been taken from a report by Professor Allen to the Cramp Company covering a continuation of the tests described in the paper, and confirming the conclusions reached with Models A and H, that "a high central cone is a desirable feature in a tube of

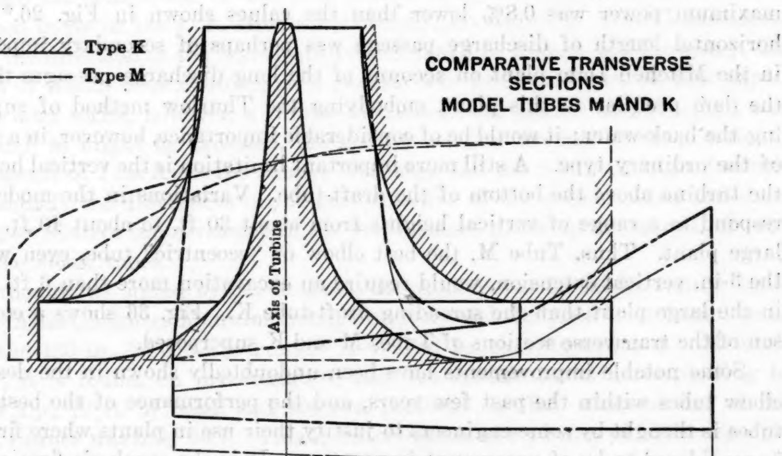



FIG. 36.

this type." This conclusion also is verified by other series of tests carried out both previously and subsequently in the I. P. Morris Laboratory. Not only are the tubes, H and K, similar, but they give efficiency curves of similar form, which are higher than those secured from any other tube. Model H, which has a longer discharge passage and requires deeper excavation, gives a somewhat greater capacity than Model K.

With regard to the termination of the original series of tests before complete data had been secured on all the tubes tested, the speaker desires to emphasize the undesirability of prematurely concluding any series of engineering experiments. The desire to set an arbitrary limit on the work can readily be understood, but until it is shown that further efforts would not alter the results, no series of tests can fairly be considered to have reached a termination. Although it is true in this instance that the additional tests mentioned serve rather to reinforce than to change the conclusions of the paper, they are valuable for this very purpose, and since they are distinctly pertinent to the subject, there is no reason why they should be omitted.

In the supplementary tests of Tube K, by re-adjusting some of the instruments higher results with the low cone arrangement were secured than are embodied in Fig. 23, these corrections amounting to from 0.3 to 0.5% better efficiency and 0.7% increase in maximum power. These results, however, did not upset the conclusion that the high cone gives the best solution.

All the models tested were intended to fit the conditions at the Mitchell Dam plant of the Alabama Power Company. It will be noted, however, that there is considerable variation in the limiting dimensions of the various models. All the symmetrical tubes except Type H conform to given space restrictions, the others do not. For instance, the curve plotted for Tube E (elbow type) was obtained with an additional horizontal extension of 3.0 ft., corresponding to an extension of 37.1 ft. in the large development at Mitchell Dam. Without this extension, the efficiency was about from $\frac{1}{2}$ to $\frac{3}{4}$% lower and the maximum power was 0.8% lower than the values shown in Fig. 26.* This horizontal length of discharge passage was perhaps of secondary importance in the Mitchell Dam plant on account of the long discharge passages through the dam peculiar to this plant, embodying the Thurlow method of suppressing the back-water; it would be of considerable importance, however, in a station of the ordinary type. A still more important limitation is the vertical height of the turbine above the bottom of the draft-tube. Variations in the models correspond to a range of vertical heights from about 30 ft. to about 40 ft. in the large plant. Thus, Tube M, the best elbow or "eccentric" tube, even without the 3-in. vertical extension, would require an excavation more than 6 ft. deeper in the large plant than the spreading draft-tube K. Fig. 36 shows a comparison of the transverse sections of Tubes M and K superposed.

Some notable improvements have been undoubtedly shown in the design of elbow tubes within the past few years, and the performance of the best elbow tubes is thought by some engineers to justify their use in plants where first cost is considered to be of paramount importance. It is the speaker's firm opinion that sometimes entirely too much emphasis is placed on first cost of that part of the plant which includes the water passages and too little on providing the best obtainable design and securing every possible saving in efficiency. First cost is, of course, an important factor, but so also is efficient operation, day in and day out, over a long period of years. Often a plant, originally built with the idea that a considerable surplus flow was available, which could not be utilized economically, has been expanded in a few years to the point where nearly every drop of water is utilized and where efficiency, at first considered relatively unimportant, has become a controlling factor. Fortunately, there are many engineers who are perfectly willing to install the best design that can be obtained whether in the turbines themselves, in the draft-tubes, or in other elements of the waterway. It is true that the spreading type of draft-tube usually involves a somewhat greater amount of form work, but simplifications are continually being made, and, even in extreme cases, the difference in cost is not an important item in the complete development. The draft-tube of the elbow type is far from simple in form; its surfaces are more complicated than

* *Proceedings*, Am. Soc. C. E., November, 1923, p. 1838.

the generated surfaces of the spreading tube, and it requires practically the same transverse space, but greater depth and down-stream length, than a spreading tube.

An examination of the various types of draft-tube described by the authors will show the marked effects which the new types of the "Hydraucone" and "spreading draft-tube" have had. The most successful of the elbow tubes are those which approach most closely to the principles of the spreading tube or those of the White Hydraucone. It will be noted, for example, that Type J, which is similar to Type H, but with the central cone omitted and the symmetry impaired by cutting off the up-stream end by a vertical transverse plane, shows the effect of this mutilation by a loss of nearly 1% in turbine efficiency.

It should be noted further that the performance curves do not extend as low as half load and that the relative performance at small gate-openings is, therefore, not shown.

Since the completion of the tests described by the authors, another series has been made in the I. P. Morris Laboratory, for the U. S. War Department, under the direction of the representatives of Hugh L. Cooper and Company, the results of which tests conform in a general way to those shown in this paper. In the later series, however, a runner of considerably higher specific speed was used and, as might be expected, the greater relative amount of the rotational components of velocity at the runner discharge and the relatively higher velocities magnify the differences in the performances of the various tubes and show a greater relative improvement in the performance given by the spreading tube over the elbow tubes.

Although, as stated by the authors, the runner was "a model runner built by the Allis-Chalmers Manufacturing Company and designed to be homologous with the 130-in., 24 000-h.p. runner furnished by that Company for Mitchell Dam", it is found, on investigation, that the specific speed corresponding to 24 000 h.p., 70 ft. of head, and 100 rev. per min., is 76.5, whereas that of the model runner was only 66.7. This is unfortunate, as a runner of higher specific speed would have provided a better basis for the purpose in view and would have emphasized the distinctions between the performances of the various draft-tubes. The speaker prefers, when possible, to test draft-tubes using turbines of a somewhat higher specific speed than will be required in a full-sized installation, as this will better insure the operation of the large unit under the varying conditions usually encountered, such as varying head. The results of the tests for the U. S. Government and the Hugh L. Cooper and Company have already been published in the technical journals and need not be presented here.

The deficiency of the model runner in specific speed is shown both in its failure to deliver a sufficient power output to correspond to the specified capacity at Mitchell Dam and in its too slow speed when operating at the point of best efficiency. The speed for unit head on which all the curves are based is 136 rev. per min. for the model, a value which is best suited to most of the draft-tubes, but which does not correspond to the speed of the large units at Mitchell Dam. Thus, it is found that the model should be operated at a speed equivalent to 148 rev. per min. under a 1-ft. head, instead of 136; or if the large unit were

to be operated at the speed corresponding to 136 rev. per min. in the model, it would have to run at 92 rev. per min., instead of 100. For the purpose of indicating the effect of such a change, Fig. 37 is presented, showing the efficiencies secured with several of the tubes at 148 rev. per min. Although at this speed the unit is still deficient in power output, the draft-tube conditions will correspond to those which would exist with a runner of higher specific speed. The curves demonstrate that the relative improvement in the performance of the turbine, due to the use of the spreading draft-tubes, H and K, is even more marked under these conditions than at the lower speed of 136 rev. per min.

Attention has been called to the progress made in the last few years in the design of draft-tubes of the elbow type. It is natural to expect that tubes of the spreading type will also be susceptible to improvement; in fact, considerable improvements have already been made and further modifications are now being developed.

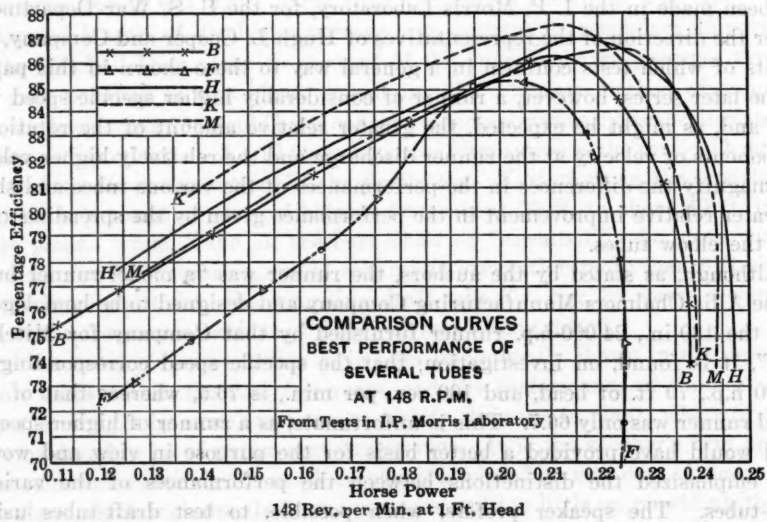



FIG. 37.

In conclusion, the speaker would summarize the results of the tests made by Professor Allen and Mr. Winter, as well as the tests for the U. S. Government, as showing that of all draft-tubes the symmetrical "concentric" or spreading type is superior to any tube of the elbow type, and that the spreading tube in its complete form, comprising a continuous annular passage surrounding a high central cone starting immediately at the runner discharge, gives the best results of any symmetrical tube.

The authors state that a comparison of definite value could be obtained only by testing the models under identical conditions, and yet all the curves are based on a speed of 136 rev. per min., which is considerably below the equivalent speed at Mitchell Dam. Likewise, all models should have the same space restrictions and yet there is so wide a discrepancy in the vertical and horizontal

dimensions of the various models that the tests are hardly comparable. Had these limitations been met, the models of the spreading draft-tube would have shown to even greater advantage.

The speaker also desires to call attention to the assistance which other power companies can render to the progress of engineering by making appropriations for experimental work similar to that for the Alabama Power Company, described in this paper. Whether this work is done by the power companies directly or by co-operation with manufacturers, it should be done, as it is necessary that engineering evolution should continue in the field of hydraulic power, in which the possibilities for further advancement are particularly valuable, and by no means exhausted.

J. F. ROBERTS,* ESQ. (by letter).†—The officials of the Alabama Power Company deserve great credit for their work in authorizing this large number of tests in order to determine the comparative efficiency of various types of draft-tubes; even more so do they deserve credit because they have allowed these tests to be made public. Although a great number of tests of various types of draft-tubes and regainers had been made heretofore by different manufacturers, never to the writer's knowledge has there been a series of tests as complete and as impartially conducted and published as those described in this paper.

In Professor Allen, the Alabama Power Company obtained a thorough, and most impartial engineer for this work. The writer has been present at several power-house tests conducted by Professor Allen, including those under discussion. Such testing is extremely delicate; and when all the variables are taken into account, the writer doubts whether it is possible to obtain accurate results within 0.25%, but feels sure that these tests were made with every possible precaution and impartially recorded.

A similar paper‡ was read before the American Society of Mechanical Engineers, in May, 1921, by W. M. White, Manager and Chief Engineer of the Hydraulic Department of the Allis-Chalmers Manufacturing Company, and commented on by George R. Shepard, Engineer for the Niagara Falls Power Company. The reason for conducting the Worcester tests is almost identical with those which actuated the Niagara Falls Power Company in 1918. The Alabama Power Company had purchased three 24 000-h.p. turbines under a manufacturer's performance guaranty, based on the use of the hydraucone form of draft-tube, and wished to satisfy itself that this was the most efficient type. Similarly, the Niagara Falls Power Company wishing to install three 37 500-h.p. turbines, and desiring to determine the relative merits of the elbow and hydraucone types,§ conducted tests, as a result of which the other water-wheel manufacturer installed concentric tubes similar in design to that of the hydraucone and obtained the wonderful turbine efficiencies of 93% from both manufacturer's units.

* Engr., Hydr. Dept., Allis-Chalmers Mfg. Co., Milwaukee, Wis.

† Received by the Secretary January 17, 1924.

‡ "The Hydraucone Regainer, Its Development and Applications in Hydro-Electric Plants", *Transactions*, Am. Soc. Mech. Engrs., Vol. 43 (1921), p. 255.

§ *Loc. cit.*, p. 287.

It is interesting to observe that from the present series of tests, the concentric tubes of the hydraucone and Moody type appear far superior to the elbow type of tube. The runner used throughout the tests had a specific speed of about 75; the Pitot tube traverses across its discharge, showed a uniform velocity with almost no angularity at the point of greatest efficiency. This combination is almost ideal for any type of draft-tube. The writer believes that had these tests been conducted with a less efficient runner, or with one from which the water discharged with a greater whirling component, the difference between the elbow and the concentric or hydraucone types would have been much greater.

The design of Tube M, as tested for the Alabama Power Company, should not be accredited entirely to the engineers of that Company, since it bears a close resemblance to the tubes which Gardner S. Williams, M. Am. Soc. C. E., designed for the plants of the Detroit Edison Company at Ann Arbor, Mich. According to the writer's understanding, Tube M was developed after the completion of the other tests, as a modification to Tube E which was furnished by the Allis-Chalmers Manufacturing Company. It should also be stated that Tube E was the outgrowth of the designs of the Allis-Chalmers Manufacturing Company, and Mr. Williams, and used extensively on the plants for which the former Company furnished the hydraulic machinery, and for which he was Engineer. A great deal of credit should be given to Mr. Williams for the development of the square elbow-type tube, which tests on Model E and Model M showed to develop such good efficiency.

D. J. McCormack,* Esq. (by letter).†—These draft-tube tests have demonstrated two things: First, that perfection has not been reached by the so-called "spreading types" of tubes; and, second, that the curved type, if properly designed, exceeds the former designs of "spreading tubes", both in power and efficiency, when using the same runner. This is obviously of great benefit to the Engineering Profession and, particularly, to power companies, since of the two the cost of building a curved type of draft-tube is much less. Further, the increased power obtainable with the curved tube from the same runner and other parts reduces the cost of total development per unit of power.

In 1916, the writer designed a curved type of draft-tube, which, when tested at Holyoke, Mass., showed $1\frac{3}{4}\%$ higher efficiency and 2% more power than the old, large radius, curved type of tube, somewhat similar to Type B. Pitot tube measurements on tubes of the large radius type showed that the water skipped the inside of the bend and set up a large whirl, with a counter-current immediately adjacent to the inner surface of the bend. The velocity at the outside of the bend was high. The new curved tube had a smaller radius at the inside, was not as high in horizontal section at the bottom of the bend, but flared out much wider at the bend, and was somewhat similar to Type E. It was found that this tube had a positive velocity at the inside of the bend of about two-thirds that at the outside, and that the velocity at the outside was much less than the previous form.

* Hydr. Engr., S. Morgan Smith Co., York, Pa.

† Received by the Secretary, January 31, 1924.

In 1919, a series of tests were made at Holyoke, to determine the effect of a shallow tail-pit and the proper outline of tail-pit for best conducting the water away from the lower end of the draft-tube. It was found that the bottom of a vertical conical draft-tube could be lowered until the throat height was approximately 0.3 of the diameter of the lower end of the tube, without any choking effect; also, that the best results were obtained when the outline of the tail-pit was symmetrical with respect to the center line of the tail-race.

In May, 1922, Mr. Karl Enz and the writer started a series of draft-tube tests at the Wellman-Seaver-Morgan Plant, which were continued until the summer of 1923. In all, about sixty different designs and types of draft-tubes, elbows, and discharge casings were developed and tested. Two of the most efficient tubes tested during the first two months were sent to Professor Allen and are noted in the paper as Tubes H and J. Since then, other more efficient tubes have been developed in this same series of experiments.

It is to be noted that the size and shape of the water passageways of both Tube H and Tube J are entirely different from any of the other tubes. The heart-shaped roof projection provides an efficient reduction of velocity in a horizontal direction, without producing eddies or other disturbances; it is designed to accommodate the natural flow of the water. The discharge is uniform and quiet at the end of the tube. Other concentric types of tubes are effective only as far as the lower end of the vertical section. The exceptionally good results obtained from these tubes attest the correct principles of their design.

The writer wishes to correct a misconception which has arisen concerning the amount of whirl in a draft-tube. The statement has repeatedly been made that a high-speed runner, particularly one of the axial flow type, has a great deal of whirl at the discharge; this is erroneous.

He has tested axial flow runners without draft-tubes, that is, forms from which the water discharged directly into the air, and also with glass draft-tubes. Under these conditions, the greatest angle of whirl measured was less than 20° from the vertical, even when operating at the lowest gate-openings. This is little, if any, more than the whirl at discharge from the runner used in these tests. Furthermore, it is generally thought that the discharge from a high-speed, axial flow runner always whirls in the direction of the revolution of the runner. This is not true. The discharge at maximum efficiency is nearly straight out and, at full gate, it has a slight whirl in the direction opposite to the rotation of the runner.

Tests made at a plant in Michigan, by Gardner S. Williams, M. Am. Soc. C. E., show that a curved tube somewhat similar to Type E (which he designed), gave a higher maximum efficiency than a tube of the concentric spreading type. Both units had the same design of axial flow runners. A great deal of credit is due Mr. Williams for being the first to depart from the old accepted long-radius type of curved tube. In 1911, he waived manufacturers' efficiency guaranties and installed his own design of draft-tube in a plant in Southern Michigan. This was followed by several others on the Huron River, each suc-

ceeding one having a shorter radius of bend in the draft-tube and a greater width at the turn, with increasingly better results.

The authors' experience in measuring the vacuum at the outside of the tube, near the band of the runner and at the center of the tube, just below the runner, bears out that of the writer on a unit having a low cone at the bottom of a concentric draft-tube. It was found to be very difficult to synchronize the unit: A piezometer placed at the center of the tube, below the runner, showed, at 0.1 gate-opening, also at 0.6 gate-opening, a pronounced pumping effect or surging at the center of the tube; one instant the water would spurt from the piezometer tube to a height of 15 ft., and the next, the tube would show a vacuum of 20 in., at the same gate-opening. A curved draft-tube, in the same plant, maintained a steady vacuum at each opening, without vibration. The writer is firmly convinced that where a cone is used on a concentric type of tube, it should extend up to the runner, in order to obtain maximum efficiency, freedom from vibration and surging, and elimination of attendant troubles.

It is important to note that the total range in maximum power is about 11%, at full gate, between the best and poorest tubes; further, that the three types of curved or eccentric tubes, J, M, and E, showed the greatest amount of power from the same runner.

In order to indicate the care exercised by Professor Allen, and the accuracy of his testing, the writer submits the accompanying curves, Fig. 38,

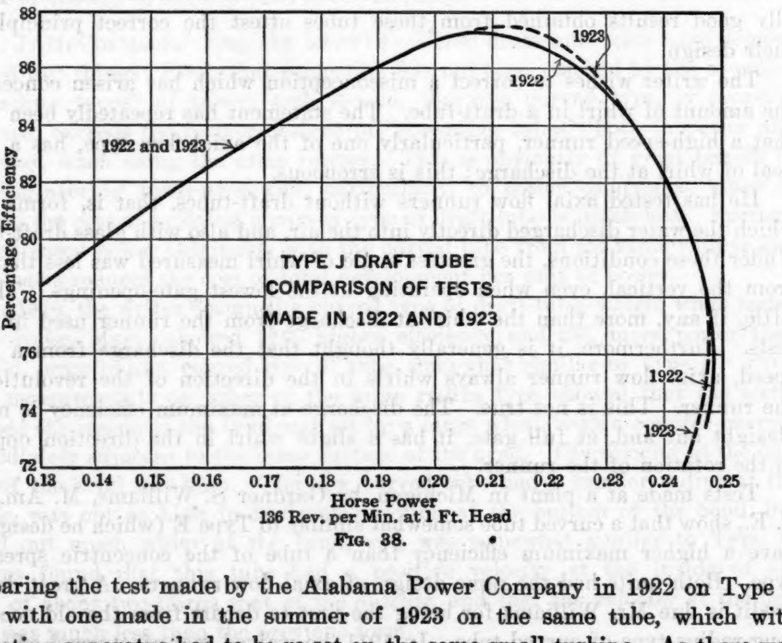



FIG. 38.

comparing the test made by the Alabama Power Company in 1922 on Type J tube with one made in the summer of 1923 on the same tube, which with all testing apparatus had to be re-set on the same small turbine. As there was only 0.4% difference in efficiency, for a small range just above the peak of the curve, and only 0.4% difference in power, at full gate-opening, it is to be seen

that Professor Allen exercised exceedingly great care in making all his measurements, particularly in view of the small amount of power to be measured,—a difficult problem.

This series of tests on draft-tubes has been beneficial to the Engineering Profession, which again is indebted to the Alabama Power Company, to Professor Allen and to Mr. Winter for the authorization and able conduct of the experiments. It is to be hoped that in the future these tests will be supplemented by further comparative results of a like nature, which course, it is believed, will receive the earnest support of the manufacturers.

E. S. Lindley,* M. Am. Soc. C. E. (by letter).†—The writer wishes to note the means of measuring the hook-gauge reading at the level of the measuring weir. The means adopted, shown in Fig. 10‡ and described on page 1 820§, is the best that has come to the writer's attention. In the method devised by the writer, a simple special gauge—home-made of sheet metal—makes it possible to save the cost of the second hook-gauge and the accurate machinist's level used by the authors.

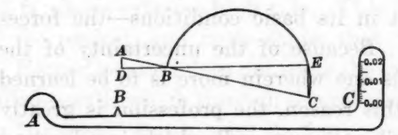



Fig. 39.

Fig. 39 shows this special gauge, the point, *A*, corresponding to the internal angle at the left of the gauge, *B*, to the upward point on the horizontal stem, and *C*, to the zero of the scale, these three points being on one straight line; the ratio, $BC:AB$, is the magnification of the gauge, which can suitably be made 5; a semi-circle is described on $BC$. To make the measurement required, the pond above the measuring weir is filled to within 0.2 ft. of the crest, the angle, *A*, rested on some point of the crest, and the point, *B*, brought to water level. $DBE$ represents the water surface, *D* being vertically below *A*, so that $DA$ is the measurement required, and *E* being on the semi-circle. $ADB$ and $BEC$ both being right angles, the ratio, $EC:AD =$ the fixed ratio, $BC:AB$; so the graduations required for the scale are marked on the semi-circle with successive radii about *C*, of the lengths to be measured multiplied by the ratio, $BC:AB$. A reading of the hook-gauge is taken at the same water surface, the special gauge gives the depth of this surface below the crest, so that the correction for hook-gauge readings is then known.

* Executive Engr., Indian Public Works Dept., Panjab Irrig. Branch, Lahore, India.

† Received by the Secretary, February 27, 1924.

‡ *Proceedings*, Am. Soc. C. E., November, 1923, p. 1827.

§ *Proceedings*, Am. Soc. C. E., November, 1923.

# OCEAN BEACH ESPLANADE, SAN FRANCISCO, CALIFORNIA

## Discussion*

By Charles T. Leeds, M. Am. Soc. C. E.

Charles T. Leeds,† M. Am. Soc. C. E. (by letter).‡—Shore protection is a most interesting phase of civil engineering, and one in which the failures—or disappointments—are probably more numerous than the successes. The forces of Nature to be resisted or controlled in such problems are so variable in character and intensity as to make solution particularly difficult. This is not a field which the novice can enter and, with the aid of a textbook or two, analyze the stresses and calculate the necessary dimensions, confident that he has achieved an economic design. The ripest experience and broadest judgment are needed, for every problem is different in its basic conditions—the forces to be considered and the results desired. Because of the uncertainty of the forces involved, this field of engineering is one wherein more is to be learned from experience than from theory. For this reason, the profession is greatly indebted to the author for so fully describing a sea-wall which is adequate, pleasing in appearance, useful, and, apparently, permanent.

The author has outlined the exposure of this locality and the numerous previous failures. It would have been interesting to know the exact location of these previous sea-walls with reference to high and low-water marks, the design and method of construction, and the probable cause of their failures. For it is axiomatic that often more is to be learned from failures than from successes.

The importance of this ocean-front boulevard to the park system of San Francisco, Calif., justified adequate protective measures. At the same time, it was essential to provide access to the beach and maintain an attractive appearance.

Undoubtedly, the chief factor in the design was safety—protection of the boulevard and the property behind it. This can be attained in one of two ways: By avoiding attack, or by putting up a strong resistance.

Father Neptune is a jealous old fellow who not only resists all encroachment on his own domain, but often makes forays beyond what is commonly considered his boundary. Therefore, the farther landward a sea-wall can be located, the safer it will be. The chief cause of sea-wall failures is the endeavor to steal from Father Neptune what rightfully belongs to him, that is, placing the wall too far seaward, in order to reclaim the maximum land area. This can only be done with a strong hand, and at great expense.

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* Discussion of the paper by M. M. O'Shaughnessy, M. Am. Soc. C. E., continued from March, 1924, *Proceedings*.

† Capt., U. S. A. (*Retired*); Cons. Engr. (Leeds & Barnard), Los Angeles, Calif.

‡ Received by the Secretary, February 18, 1924.

In the present instance, this factor of location seems to have been well considered, and the structure placed as far landward as was consistent with adequate width for the boulevard. Doubtless, a lighter wall would have been possible, if placed farther back, but the value of the land necessary would probably have been greater than the amount saved in wall construction. Conversely, more land might have been reclaimed by placing the wall farther seaward, but at a much greater hazard and a consequent necessity for stronger design.

Comparison of the cross-section of this wall and the relative elevation of high-water level with the details of some of the best examples of sea-wall construction in Europe, such as Clacton, Bridlington, and Blackpool, England, or Scheveningen, Holland, is instructive. It will be found that in these older types, the high-tide level is well above the toe of the wall. In the case of the San Francisco sea-wall, however, it is noticeable that the toe of the slope is between mean high and extreme high tide, and the wall has been placed sufficiently landward so that the normal beach line is half-way up the concrete revetment. The toe of the sea-wall proper, and the top of the bleachers is 8.5 ft. above extreme high tide. This is no mean factor in assuring the safety of the project, and truly shows that "discretion is the better part of valor."

Proper location does not solve the entire problem, however. Many walls have failed—walls which through several years have appeared to be well located, until an "unusual" storm, with its fierce, on-rushing waves, has driven its forces far landward of the usual limits. Then, no matter how heavy the wall, if it presents a straight, unvarying front to the attack, the situation is perilous in the extreme, for the on-rushing wave is thrown upward to a height proportional to the square of its velocity. If this wave falls at the toe of the wall, it scours out material, and, in retreating, carries the eroded material with it.

In the effort to prevent this, some sea-walls have been given a curved cross-section, as at Galveston, Tex., in order to deliver the back-wash approximately parallel to the beach slope. By extending the upward curve sufficiently to form a coping, the wave is returned on itself and prevented from falling behind the wall. This purpose has been well accomplished in the San Francisco wall. Even though the curve at the toe of the wall is made tangent to the slope of the beach, and the apron extended, there is still likely to be undue scour. At Galveston, it was necessary to place considerable rip-rap in front of the wall; often an apron extension has to be added, as at Scarborough, England, and Scheveningen, Holland.

Rather than attempt to resist the waves so directly, it may be wiser to remember Napoleon's adage, "divide and conquer." If this can be done, the battle is more easily won. A good illustration is the sea-wall at Coronado, Calif., which consists simply of a light concrete retaining wall, with an ample supply of rip-rap in front. The whole sustains and protects the boulevard along the ocean front. It is an excellent example of an economical, permanent, and substantial type of protection, and occupies no great area of beach space. If any settlement should occur, due to an unusually severe storm, more stone can be added on top, and the wall will be stronger and better than before, and no labor or material will have been wasted. Of course, an important feature of the design is the proportion and size of the rip-rap used.

As regards economy, security against destruction, and ease of repair, this type of construction is probably superior to that at San Francisco. It can lay no claim to beauty of appearance, however, and is also lacking in convenience of access to the beach. The latter objection might be removed by the construction of concrete stairways to the beach, but the former objection is inherent in the design.

The multiplicity of irregular surfaces which rip-rap exposes to the waves, does not stop them abruptly, but splits and turns them in various directions and "by the formation of eddies and air pockets, dissipates their force." This is much more efficiently accomplished than by any construction of regular shape.

Nevertheless, for convenience to the public and for the sake of appearance, some regular form of construction is usually preferable. By making it in the form of steps or bleachers, as was done by the author, the same purpose of dissipating the wave force has been effectively accomplished and, at the same time, a structure of maximum convenience to the public and of good appearance is obtained.

The flatter the slope of these "bleachers" the greater will be the intrinsic stability of the protection, and, therefore, the cheaper the cost. The "bleachers" constructed some years since at Revere Beach, Massachusetts, are an excellent example of this; it is hoped that a description of them may be presented.

The absence of rock or clay strata on which to found the wall, or in which to drive the sheet-piling at the toe, is a difficulty which has been well considered in the design. Also, the use of pedestal piles under the wall proper seems to be an excellent feature, since the back pull of waves is too often ignored.

The use of clay packing under the bleachers "to provide an impervious blanket and prevent the removal of any of the confined sand by seepage, in case of a crack in the bleacher section itself", is a precaution not always taken. An instance of this came to the writer's attention a few years ago. A sea-wall of concrete piling had been constructed, but the joints proved to be faulty. The back-wash of the waves soon sucked out much of the sand backing; this allowed the concrete pavement above and behind the sheet-piling to collapse; waves breaking over the top then produced an excessive pressure behind the sheet-piling, causing failure in several spots; and this, in turn, gave the waves direct access to the rear of the sheet-piling. As there were no cross-walls, the whole structure eventually collapsed.

The precaution of maintaining a standard cover of 3 in. of concrete over all reinforcement is excellent. The writer believes that failure of reinforced concrete in sea water can more often be traced to insufficient cover over the reinforcement, or to porous concrete, than to any other cause.

In conclusion, the writer desires to congratulate the author for the excellence of a design that so well combines the favorable elements of a sea-wall having a curved face, with those of concrete bleachers, for adapting the design to local conditions, and particularly for locating the structure advantageously with respect to the high-water mark. Not the least valuable part of the paper is the cost data contained in Tables 1 to 5.*

* *Proceedings*, Am. Soc. C. E., November, 1923, pp. 1857–1858.

# BEACH EROSION: ITS CAUSES AND CURE
## Discussion*

By Messrs. Charles W. Staniford, Victor Gelineau, E. J. Dent, Merton C. Collins, Charles S. Riché, and Henry J. Sherman.

Charles W. Staniford,† M. Am. Soc. C. E. (by letter).‡—The examples of the effectiveness of a lateral outshore method for the protection of ocean beaches, given by Mr. Ripley, are interesting in that he clearly defines the almost universal manner in which the destructive elements function; his remedy, however, appears to be too general and vague, unless a more definite end or objective is first known. In other words, an endeavor may be made (1) to stop erosion on an advanced line with a desire to regain land; or (2) to retain the beach on the advanced line where the attack is so vicious and intense that it appears to be necessary to hold this line, and where no rear line is available and further erosion would wholly destroy the beach; (3) simply to hold the beach on a general normal line, and maintain it as a bathing beach.

In the first two cases, the results have usually been very unsatisfactory, although the attempts often have been undertaken with apparently well planned structures and at great expense. In almost every instance, due to the question of cost, rather than the real judgment of the engineer, these formidable structures have been rendered ultimately useless; because so much expense was involved in their construction above ground, the necessity for properly founding them was too often overlooked. Many of them have been built of reinforced concrete—beautiful structures—which failed through such neglect. Even if founded on wooden piles, they could have been made permanent at low cost.

An extensive reinforced concrete jetty recently examined by the writer, in which complete destruction is impending, is a good illustration of this condition. To obtain absolute permanence, a heavy and apparently fairly well designed structure was built of reinforced concrete. After about ten years, the struts used to stabilize the design, including the piles, showed failure due to steel exposure and consequent cracking and peeling of the concrete. This condition was occasioned by an erosion on the face, causing increased head and pressure in the rear. In other words, because it was almost impossible to force the great pile sections down to a sufficient depth, the consequent instability of the foundation permitted a movement sufficient to destroy the integrity of the concrete and the steel. There is no question but that wooden construc-

* This discussion (of the paper by Henry Clay Ripley, M. Am. Soc. C. E., published in January, 1924, *Proceedings,* but not presented at any meeting of the Society), is published in *Proceedings,* in order that the views expressed may be brought before all members for further discussion.

† Cons. Engr., New York, N. Y.

‡ Received by the Secretary, February 5. 1924.

tion would have been much cheaper and could have been placed at a sufficient depth to ensure stability for many years to come.

It appears, therefore, that the structure proposed by Mr. Ripley would be costly and difficult if built for permanence beyond the low-water line. In fact, such structures have always been difficult to build even well above high water. If it should become necessary to use this method, reinforced concrete as a material would appear to be out of the question; derrick stone with or without a wooden core might be tried, but would be expensive.

The author states that placing the breakwater outside the low-water shore line will cause accretion inside, and he states elsewhere that it will leave the beach unobstructed for bathing purposes. This is a feature of shore protection that the writer wishes to emphasize, namely, the maintenance of bathing beaches in their present position, and this discussion provides an opportunity to define the status of beaches and methods for their protection.

Long ago, the coast line of the United Kingdom, subject as it is to attack from all directions, gave concern, owing to the alarming amount of erosion taking place. On account of the continual increase of population and a consequent demand for recreation, bathing beaches were established. The erosion, carefully surveyed and reported at frequent intervals, began to show such comparatively large areas of loss against a limited amount of upland that it was easy to obtain public aid in instituting protective works to hold the shore lines or retard the destruction. With these general protective structures, resorts, supported in reality by bathing beaches, were established and also protected, the structures increasing in magnitude and permanence.

During this early period, the great coast line in America was also receding, but since the amount eroded was so small in comparison with the vast areas in the rear, it caused no alarm. In due time, however, an increasing number of people seeking recreation were drawn to various points on the ocean front, and then the ravages from erosion were observed with an ever-increasing conviction that the natural tendency toward destruction must be combatted with artificial works.

Thus, it is only within recent years that any particular attempt was made to protect the ocean front permanently; the early structures were small efforts undertaken under various and different theories of control and built of many materials. The report of the Board of Commerce and Navigation, State of New Jersey, for 1922, shows that the total erosion since the first records (survey of 1839), has amounted in some places to several hundred feet.

The Long Island beaches which are the bathing places for the great population of the City of New York, have also been subject to a continuous erosive attack. On account of the small length of beach involved, together with the great demand, works of considerable cost were finally undertaken, in order to arrest the destruction. In some places, the final effort was in reality a vindication of Mr. Ripley's theory, but these vast outshore lateral breakwaters, although they were barriers against absolute destruction, did not maintain the beaches for bathing.

It is the bathing feature of these beaches that is the magnet which attracts a vast population between the Atlantic seaboard and the Middle West. It is

this magnet which has created practically a continuous seaside resort from Sandy Hook to Cape May, N. J., a distance of 120 miles. This recreational growth has included the building of cities, boardwalks, bridges, and splendid residences, culminating in Atlantic City, the finest ocean-side resort in the world. All this has been built directly contiguous to the ocean, and has been made possible by a comparatively cheap method of beach protection, consisting in the main of jetties or groins extending from inside the high-water line, or natural bank, outshore to some point below the low-water line. In some cases, bulkheads have been built between the jetties, and, in general, these works have been successful, except where too ambitious plans were attempted by locating the bulkheads too far from the shore with, perhaps, the idea of regaining land.

At first, in New Jersey, no particular incentive existed for any endeavor to arrest the erosion, but with the establishment of pleasure resorts dependent wholly on their ability to afford bathing beaches, pioneer works were commenced in a crude way, gradually increasing in efficiency to the present time. Particular attention is called to the methods now used, as these beach fronts must be retained where they are, and the work, although being done efficiently as outlined, is not burdening the private owners, or the State, with an undue capital charge. If the method proposed would retain the beach for bathing and could be built to be permanent, without excessive cost, it should receive serious attention. The prevailing methods are the result of the examinations and plans of many engineers. The only question is as to the angle or curve which these projecting structures should follow in order to obtain the best results under peculiar local conditions.

The writer will confine his remarks to normal beach-front erosion rather than to the peculiar conditions at inlets, of which New Jersey has many. Through them, the waters from extensive interior bays pass into the ocean. Each inlet is an independent study, requiring structures sometimes of peculiar shape and position, which often have a certain influence on contiguous beaches.

It is fair to assume that the part of the New Jersey beach-front, facing the ocean along a narrow peninsula, and widely separated from the real mainland, would have continued to lose by erosion until finally destroyed. The beach is being held on what is left of this narrow neck of land by the simple methods described, therefore, any radical change in these methods must not call for any great increase in cost.

It is generally recognized by all who closely watch beach conditions that the offshore bar which so often exists parallel with the beaches, performs a helpful function in aiding to maintain the bathing beaches with so simple a method of construction. Undoubtedly, this same offshore bar has always accompanied the shore line as it retreated, but until protective structures were also used, the erosion did not stop.

The writer agrees with the author that beach problems have been fraught with disappointments; hence, before adopting any new and expensive method, it must be shown to be effective over those methods which are now satisfactory as well as cheap. The importance of the subject may be emphasized by the fact that for the year 1922 the New Jersey beach properties were assessed on

a valuation of $291 000 000; and, in addition, a valuation of $18 000 000 was fixed exempt from taxation, or a total of $309 000 000.

In order, therefore, to state the case clearly, and as a vindication of the methods usually followed in the maintenance of straight beach fronts, the following is submitted:

*First.*—That the principle of a lateral breakwater beyond low water is not tenable for the Atlantic Coast without undue expense in construction.

*Second.*—That such construction is not necessary for the maintenance of beaches for bathing.

*Third.*—That the offshore bar provided by Nature, operates naturally in performing some of the functions of such a breakwater.

*Fourth.*—That no attempt should be made to regain land by lateral works beyond the natural shore line.

*Fifth.*—That most failures in the past have occurred to monolithic reinforced concrete structures designed to gain beach instead of to retain it.

*Sixth.*—That the maintenance of the shore line by the jetty system properly designed, placed, and founded, with the few necessary connecting bulkheads correctly placed, is fairly successful.

*Seventh.*—That works for sea-coast and beach protection constructed in reinforced concrete as a rule have not been successful. Admitting, first, that the product has been perfectly made—this is frequently subject to grave doubt—failure has occurred through movement due to the utter impossibility of obtaining proper penetration of piles or sheet-piles into some materials without undue expense, and the resultant disturbance sufficient to destroy the integrity of the combination, exposing the steel, with the consequent failure of these costly structures. That wooden piling and timber (treated when subject to attack from marine borers) may be made to attain the required penetration and, in combination with large rock, may be designed to produce a greater life at much less cost. That such structures, when properly founded, being little exposed and always wet, may serve their purpose even if renewed after a reasonable life, with less capital charge against them than some of the formidable monstrosities which have been built for supposed permanence, but which, in a few years, have become absolutely worthless.

VICTOR GELINEAU,* M. AM. SOC. C. E. (by letter).†—This paper well explains the major causes of beach erosion, namely, the pounding of heavy seas and the transportation of sand by currents alongshore. High winds blowing from the land also have a considerable effect at times, particularly on exposed sand-spits. The proposed remedy, however, may not prove to be of universal application. In any event, its adoption would require careful consideration.

The author cites two examples, one in France and the other in Brazil, to prove the principles submitted. The appraisal of the value of these two examples in their possible application to other situations, however, depends on further information as to the conditions existing at those sites.

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† Received by the Secretary, February 9, 1924.

Different beaches possess widely differing characteristics. Among the factors which must be appraised before undertaking the treatment of any beach are the following: Height of waves, particularly during storms; range of tide; presence or absence of sand dunes; proximity to inlets (and this may involve a study of the tidal prism and the form of the body of water supplied by the inlet); gradient of the beach and of the ocean floor immediately offshore; existence and form of offshore bars and of bulkheads, or other structures situated outshore from the high-water mark; direction of the prevailing drift; and character of the material forming the beach.

The writer agrees with the author that the primary cause of erosion is the alongshore current, but the destructive effect of heavy storm seas is highly important in itself, as they agitate the sand, which the alongshore current is thereby enabled to transport. Wherever the beach exists in a natural state, that is, without bulkheads, groins, or other obstructions below the line attained by storm seas, and the alongshore current is slight or practically non-existent, it is usually safe to assume that the beach will not readily undergo permanent change. Fluctuations will occur, but the effect of one set of conditions in wearing down a beach will be offset, or practically so, by the operation of an opposite set of conditions, and the resultant changes will be very slight. On the New Jersey coast, for example, a beach may be worn down by northeast storms and built up again after long periods of prevailing southwesterly weather.

On this coast, the primary cause of alongshore currents having a relatively high velocity is found in the inlets through which flow the tidal waters that supply the bays and sounds. Every change of level between the waters of the ocean and of these bays generates currents, with a consequent effect on the beach in the vicinity of the inlets. With northeast storms, for example, particularly at the spring tides, the general level of the ocean is raised, and, in addition, heavy seas roll far up the beach. These seas loosen the sand which is then carried in suspension, and, encountering a flood-tide into an inlet, is quickly transported from the beach and deposited in the relatively still waters of the bay.

By his plan, Mr. Ripley seeks to obtain the beneficial effects of a shelving beach. One danger, however, is its probable failure to prevent the alongshore current that it might generate. At points near inlets, great care would be required in locating a breakwater, as it could readily guide the tidal currents in such a way as to greatly increase their velocity, with resulting erosion of the beach instead of protection. By preventing free re-adjustment of the levels of the water immediately inshore and outshore of the breakwater, it might create eddies or currents which would inpinge on the beach. Serious shifting would certainly result if there were nothing to check the alongshore currents which were thus generated.

Another difficulty in applying this breakwater theory to many of the New Jersey beaches would arise from the necessity of avoiding any obstacle to bathing. The greatest natural attraction of these beaches to enormous numbers of people lies in their splendid bathing facilities. The first consideration at

these resorts is the preservation of the bathing beach. In fact, in a neighboring State, recent years have witnessed the rebuilding of an eroded bathing beach by the expedient of dredging sand from a bay back of the beach.

Another objection to the offshore breakwater would be its relatively high cost. If the depth of water were considerable, the wall would have to be of large cross-section. For example, assuming a top width of 15 ft. and a height of 15 ft., with side slopes of 1 on $1\frac{1}{2}$ (and the outer face should be flatter than this), there would be approximately 21 cu. yd. of rock per lin. ft. of wall. Estimating rock at $12 per cu. yd. in place, this wall would cost approximately $250 per lin. ft.

The rock used in such a structure would need to be in large units, and even so, the impact of heavy seas, with more or less shifting of the sand at the toe, would dislodge some of the wall material. Therefore, it would be necessary to replace these dislodged units at intervals, in order to obtain the maximum efficiency. To reduce this battering effect, a flat slope could be adopted on the offshore face, but this would call for a greatly enlarged cross-section with a corresponding increase in cost.

The example taken from the French coast is described as being completely connected with the shore at one end and partly so at the other. From the description, it is believed that the opening is at the windward end of the breakwater and that this forms in effect a sand trap. It is not clear what caused the sand to collect outshore from this wall.

The difficulties in protecting sandy beaches have been primarily of a financial rather than engineering nature. Erosion can be checked by means of heavy key structures, and for many places where erosion is severe, the proximity to shallow bays would render it a simple matter to build up a beach by hydraulic dredging. This would require structures to prevent repetition of erosion. In many instances, the strip of beach threatened is not worth the cost of adequate protection works. In addition, until recently, no general control was exercised over the efforts of landowners or municipalities to protect their beaches, and, in many instances, structures built according to the varying ideas of different owners operated to conflict with one another. The problem usually becomes complicated where protective structures, such as groins, bulkheads, etc., have been erected and have proved to be ineffective. Here, the situation is complicated by continued erosion, eddies, back-wash, or other unfavorable conditions, caused by these structures, with no nucleus of beach on which to build. Public control should eventually be extended, if possible, to the point of marking out a zone along the beaches, with a view to preserving a sandy strip between the ocean and the non-marine structures, such as roadways, dwellings, etc. Piers, groins, and other structures extending beyond the high-water mark should then be designed with a view to having no destructive effect on neighboring properties. It seems certain that where a tight structure is built on the beach and is successful in gathering sand, resulting erosion will occur leeward of the intercepting structure, as the eroding forces will continue to operate at this leeward point and the sand which normally would preserve the balance between erosion and accretion, is prevented from flowing to this unprotected point of attack.

In general, it is believed that sandy beaches can be protected by measures much less expensive than those proposed by Mr. Ripley, unless the erosion has reached important structures or threatens the safety of an entire community. In such an event, large key-works, designed to stop the erosion without delay, will be required, but even then it would seem that due consideration should be given to the possibility of extending structures from the shore. These structures can be designed so as to check the battering from the seas, and, at the same time, trap the sand. The writer believes that the existence of bulkheads or walls running approximately parallel to the shore and below the high-water mark, presents one of the gravest difficulties in protecting a beach, and it is very doubtful in such a case whether even the expedient suggested by Mr. Ripley would have any beneficial effect, unless the possibility of scour from alongshore currents was definitely precluded by building tight groins from the shore to the breakwater. These would add greatly to the entire cost of the project, and, in many instances, would be unsuitable if designed to preserve bathing beaches. Presumably, also, these groins would have to be built successively from the leeward end of the breakwater to build up the area between the wall and the shore.

The writer has conducted experiments at a point on the New Jersey coast where serious erosion has been in progress for many years. As a protective measure, a concrete sea-wall, about 3 500 ft. in length, was built on the ocean front, just above the low-water mark. The southerly end of this wall is just north of the north spit of a large inlet. As a part of the work, a series of low timber groins were extended out at right angles from the sea-wall. This combination of sea-wall and groins not having proved as successful as was expected, a heavy rock breakwater jetty of large cross-section was built from the southerly end of the sea-wall, extending oceanward at right angles. A sand-bar lay a few hundred feet offshore and approximately parallel to the beach. It was expected that the jetty could be built out to this sand-bar, and that, in consequence, the alongshore current would be immediately checked and the sand-bar would then move inshore. These expectations, however, met with disappointment. The offshore bar did not furnish an exact analogy to Mr. Ripley's breakwater, as it was covered at all stages of the tide. It was very effective in forming and guiding an alongshore current at flood-tide, which had an extremely powerful effect in wearing away the north point of the inlet. The stone jetty never reached the offshore bar for the reason that the latter retreated with the extension of the jetty. Consequently, quantities of rock far beyond those originally contemplated were required to build this jetty, on account of the rapid deepening at the outer end as the work progressed. A deep channel formed where the bar existed at the beginning of construction.

At flood-tide, floats placed at the intersection of the northerly (windward) side of this stone jetty with the face of the concrete sea-wall, moved with a high velocity oceanward, along the edge of the rock jetty, sweeping around its end and into the channel of the inlet. This is a fair analogy to the V-shaped bays at the inland end of which the tidal range is so much greater

than in the open sea. It is extremely difficult to avoid or control the flood-tide current in the vicinity of inlets. Offshore bars have a pronounced effect on the direction and velocity of these currents which are frequently so destructive.

E. J. DENT,* M. AM. SOC. C. E. (by letter).†—Large sums of money have been spent in the United States in connection with the protection of its ocean beaches, especially those along the south shore of Long Island and along the coast of New Jersey. In carrying out this work, the usual practice has been to follow the rules laid down by British engineers for the protection of the beaches along the coast of England. Not enough consideration has been given to the fact that the character of these English beaches and the tidal and other conditions along them are radically different from those prevailing in the vicinity of New York City.

Along the New Jersey and Long Island beaches are numerous examples of bulkheads built at about the high-water line, and of sloping groins extending from this point to a little beyond the low-water contour. An examination of these existing structures will show that, in many instances, they have failed to function, and that the ordinary statement of principles involved affords no explanation of these failures. A clear and convincing summary of the theories applicable to such cases, including a description of the errors that have caused the failures, or a frank admission that these methods of beach protection are not applicable to the particular locality, would be of immense value to engineers and their clients.

It is an observed fact that groins do not always protect a beach. The author gives his reasons for believing that groins increase beach erosion instead of reducing it. The writer came to a similar conclusion several years ago,‡ and has had no reason to change his views since that time. Groins occasionally appear to function in the manner usually claimed for them; but the writer believes that these appearances are deceptive, and that, in many cases, the accretion that follows the construction of the groins is due to other causes, and takes place in spite of their construction instead of on account of it. Mr. Ripley is to be congratulated for calling attention to a system of beach protection that will positively and definitely stop erosion whenever the protection afforded is worth the cost of obtaining it.

The objections to offshore breakwaters include: (*a*) the cost is frequently excessive, as compared to the value of the property protected; (*b*) an open beach is more attractive for a shore colony than one along the shore of an inner harbor; and (*c*) the interruption to the movement of littoral drift may damage adjacent beaches. Items (*a*) and (*b*) require no further comment; but Item (*c*) may properly be discussed at greater length.

The author's description of wave action might lead to the conclusion that waves normally move the sand from the outer line of breakers toward the beach, and that, if it were not for littoral currents, wave action would always

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† Received by the Secretary, February 9, 1924.

‡ "The Preservation of Sandy Beaches", by E. J. Dent, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXXX (1916), p. 1786.

result in accretion. It is a fact that, to a certain extent, shallow-water waves are waves of translation. Within the outer line of breakers a certain quantity of sand may be thrown into suspension and may be found near the surface of the water, and such sand will be carried slowly toward the shore as the result of the landward movement of the surface filaments. It is, however, also a fact that, for every gallon of water moving toward the beach, there must be another gallon of water moving away from the beach. This offshore current is the well-known undertow, near the bottom, where the load of suspended sand per gallon of water is much greater than the corresponding load near the surface. The net result of all wave action is the transfer of material from the shore to deep water in the ocean. The wave-cut terraces and the marine deposits of débris just outside those terraces are common land forms, recognized by the geologists. If any one should care to investigate the matter, the eastern end of Long Island affords a remarkable illustration of a terrace that has been formed in the most recent geological times and that is to-day in the process of growth.

Assuming for the sake of discussion an east and west beach, extending continuously for several miles, there will normally be a movement of sand (beach-drifting) in the direction that the winds and waves at the particular moment dictate. A change in the direction of wind may reverse the direction of the beach-drifting. A preponderance of wind coming from one direction may cause a corresponding preponderance of beach-drifting.

In Fig. 2, an offshore breakwater is indicated as having been built for the protection of a section of the beach just described. In the rear of this breakwater, wave action will have been stopped, and beach erosion will also have been stopped. In the rear of the eastern end of the breakwater, a heavy

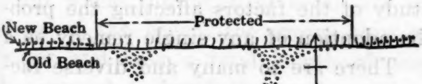


FIG. 2.


FIG. 3.

accumulation of sand is shown. This sand will be brought to this point by beach-drifting, but, as a result of the protection of the breakwater, it cannot be carried beyond that point. In the rear of the western end of the breakwater, a similar but lesser accumulation of sand is indicated. Sand accumulating in the rear of the two ends of the breakwater is taken away from the general circulation along the beach, and erosion of the adjoining beaches results.

In Fig. 3, another method for protecting a section of sand beach has been indicated. This method consists in the construction of a solid pier, perpendicular to the shore line, and extending from about the upper limit of wave action to between the 10 and 30-ft. depths, depending on local conditions. The pier should extend into water deep enough to prevent wave action from greatly disturbing the bottom, as otherwise there would be a tendency for the sand to follow along the pier and pass into deep water near its

outer end. With such a pier in existence, beach-drifting will cause an accumulation of sand in the angles near the shore end of the pier. These accumulations will protect the beach in their rear as effectively as an offshore breakwater. The accumulations along this pier are exactly similar to those found in nearly all cases where harbor entrances have been protected by jetties extending to relatively deep water.

It will be recognized at once that the construction of the pier shown in Fig. 3 will cause erosion of adjoining beaches. The extraordinary feature of the case is, however, that it is rarely recognized that the fixation of identically the same sand in identically the same locality by some other process will also cause erosion of the adjoining beaches. The fact of the matter is that, on any beach such as the one described, the quantity of sand in circulation is limited, and to take any part of that sand out of circulation necessarily depletes the remaining sections of the beach. The question whether the sand so fixed in position is worth more after being stabilized than it was while circulating along the entire beach, determines the question as to whether the measures taken have resulted in benefit or loss.

The State of New Jersey has undertaken an extensive investigation of its beaches. It is to be hoped that the information gained will place this entire subject on a firmer foundation than at present. In the meantime, papers such as that of the author serve a valuable purpose in focusing the attention of engineers on the essentials of the problem, and in guiding future investigations into the most promising channels.

MERTON C. COLLINS,* ASSOC. M. AM. SOC. C. E. (by letter).†—The writer concurs with the author's statement that the efforts to prevent erosion of beaches have been fraught with many disappointments, and believes that these failures are chiefly due to insufficient study of the factors affecting the problem. Exception is taken, however, to the adoption of any single remedy as a panacea for all beach-erosion problems. There are so many and diverse factors which enter into beach erosion that, although the theory expounded by Mr. Ripley is extremely valuable to the designer of beach-protection works, he must needs study each particular case and adapt his design to local conditions.

With its three thousand odd islands, the beach perimeter of the Philippine Islands is perhaps greater than that of any equal area of land and its beach-erosion problems are in like proportion. However, the comparatively low value of the land and lack of sufficient funds make it impossible to provide remedies, except in extreme cases where the erosion is seriously threatening some community. Such a case was brought to the writer's attention in 1921, while he was engaged on provincial port works for the Philippine Government, and is mentioned because of its direct bearing on this paper. Although the method used in this case was not along the lines of Mr. Ripley's suggestion, it is believed that it was the best solution under the circumstances. At last reports, it was highly successful and was the means of building up a

* Ross, Calif.

† Received by the Secretary, February 13, 1924.

beach, the erosion of which had for years been threatening the Town of Aparri.

Aparri is situated on the northern coast of Luzon and is subjected to an annual season of severe typhoons, and also to a continuous littoral current which has been eroding the shore line and destroying valuable property. Various attempts at preservation have been made in the past, chiefly by constructing timber groins at right angles to the shore line.

From study by the writer, including the history of past attempts to stop the erosion, it was found that the resultant direction of prevailing winds and littoral drift was S. 70° W., which made an angle of about 20° with the shore line. All previous groins had been constructed at right angles with the beach, the result being that the current was deflected onshore, causing erosion on the "weather" side of the groins and their subsequent destruction. It was decided that the proper angle which the groins should make with the shore was 110°, or at right angles to the resultant direction of the prevailing winds and littoral drift. Any less angle would result as before, and any greater angle would cause an offshore current and erosion on the "lee" side of the groins, a condition aptly described by the author with reference to an artificial obstruction extending far out from shore. The writer differs with Mr. Ripley in that he believes that the angle that such an obstruction makes with the littoral current greatly affects the resultant erosive action.

The groins at Aparri, which are placed parallel to each other at 328-ft. centers, consist of grooved concrete piles driven on 10-ft. centers, between which wooden sheet-piling is driven flush with the ground. Three horizontal flash-boards are placed above the ground line, to which others are to be added as the beach builds up. Each groin extends from the uneroded beach to a point where the depth at mean low water is about 2 ft. Owing to the flat slope of the beach, the groins thus extend about 160 ft. beyond mean low water and have a total length of about 300 ft. each. The flash-boards are low enough to break the erosive action of the current without causing undue turbulence.

In the case of the Aparri beach protection, the question of using a remedy such as that suggested by Mr. Ripley was out of the question, owing to the local lack and prohibitive cost of procuring material from a distance for such a breakwater, and also due to a local factor common to most Philippine beaches, namely, that the populace depends a great deal on fishing and small-boat transportation, and having always used the beaches for landing, such a submerged breakwater placed parallel to the shore would prove extremely dangerous to their small craft.

Charles S. Riché,* M. Am. Soc. C. E. (by letter).†—To one familiar with the beach of the Gulf of Mexico at Galveston, Tex., this paper is of much interest.

The hurricane of September 8, 1900, caused much erosion along this beach; in some places the shore line was cut back nearly a city block. Thereafter, there was a gradual accretion, but not enough to restore the original shore line,

* Col., Engrs., U. S. A., St. Louis, Mo.

† Received by the Secretary, February 15, 1924.

which, in fact, had been eroding in places prior to 1900. Between 1902 and 1904, a concrete sea-wall was erected about 200 to 300 ft. behind the existing shore line, leaving this width of beach.

In August, 1915, another hurricane as severe as that of 1900 occurred, during which waves of great size and force dashed against the wall. The spray which rose 40 ft. and more into the air, was heavily charged with fine sand, many thousand yards of which were blown back of the wall causing deposits from 1 to 3 ft. in thickness on near-by streets.

Along most of the water-front, the subsidence of the storm left the sea-water lapping the toe of the wall, so that immediate measures were necessary to protect its pile foundation from the teredo. The beach no longer existed; it had been thrown over the wall.

Accretion thereafter was slow and disappointing. This beach had been an asset to the city for sea-bathing and resort purposes, and, therefore, its restoration was greatly needed and desired.

About two miles to the eastward of the city are the jetties at the harbor entrance. In the pocket, south and west of the nearest jetty, an enormous accretion has taken place since 1915, but along the front of the city there has been little change; the sand in suspension seems to drift by and finally bring up in this pocket back of the jetty. The protection of the jetty prevents its return.

Eventually, it is believed, this pocket will fill with sand so that the area of accretion will extend along the front of the city, but this will require many years and many million yards of sand. Meanwhile, the problem is to restore the beach—at least, in spots—to some approximation of its former extent, and without the long delay which would result from waiting for the filling of the pocket behind the jetty.

Small jetties or groins have been helpful in this locality, but seem to be limited in effectiveness. They should be practically water-tight, or the fine sand will "flow" through them. Usually, there has been a deposit of sand on each side of these groins at their inner ends, but mostly on the prevailing "weather" side.

Mr. Ripley's suggestion of a breakwater parallel to the beach and a few hundred feet offshore impresses the writer as promising a greater effectiveness than a series of groins. This breakwater would probably interfere somewhat with the smoothness and regularity desired in a bathing beach, but so also would the groins. It may prove advisable to make occasional connections between this breakwater and the shore.

To make such a breakwater fully effective, it should be practically water-tight and strong enough to resist storm action. These requirements, unfortunately, would result in high cost. Double or triple lap sheet-piling would be quite effective for this purpose. This would require considerable penetration and, perhaps, some rip-rap protection to be secure against storms.

Unfortunately, the warmth of the water in the Gulf of Mexico results in rapid destruction of exposed timber by the teredo. Any timber piling, therefore, would have to be heavily creosoted—an expensive procedure. Interlocking steel sheet-piling would probably be still more expensive and, possibly, no more

permanent, by reason of corrosion. Reinforced concrete piling might prove the most advantageous.

A trial of Mr. Ripley's plan would be interesting and promises good results.

HENRY J. SHERMAN,* M. AM. SOC. C. E. (by letter).†—The experience and reputation of Mr. Ripley in harbor and coast protection are guaranties that his paper will be studied by all those interested in this subject, about which too little is known. To go a little further than the author, it might be stated that in no branch of engineering have the disappointments been so numerous and so persistent. Nevertheless, during the last quarter of a century the writer has observed distinct progress in beach protection along the North Atlantic Coast, principally on the Long Island and New Jersey Coast—due to the fact that some, but not all, of the laws controlling wave and current action are known.

Until a few years ago, many devices were designed and built by a local rule-of-thumb man with little or no knowledge of the actual conditions or the forces he was attempting to control. They were structurally inadequate, even though well placed in location and direction. Again, many of the failures along the New Jersey coast have been due to the proximity of the structure to the shifting channel of an inlet, unsuitable type, failure to tie in at the shore end, insufficient size and depth of round piling and sheet-piling, poor fastenings, besides a variety of other causes. Then, too, in characteristic American fashion, there has been gross neglect of maintenance. Jetties which acted as good sand catchers while in repair, soon lost all they had gathered when breached.

In describing the action of an artificial structure in the form of a low jetty, the author states that there will be erosion on the lee side. If the lee side is protected by stone, the wash of the current is broken up and quite frequently a deposit occurs near the structure on a gently sloping foreshore. Farther away, however, as the author points out, one must expect erosion and prepare accordingly. In the protection of property of great value, where the loss is along a front of little value, this objection is of minor importance. In fact, the writer overheard an eminent authority say the time might come when it would be necessary to give up beach of low value to protect an adjoining section of high value. Certain it is, that erosion is constant and that the deficiency must be supplied.

The author is correct in stating that there is erosion at the outer end of a jetty. Designers of jetties attempt to guard against this undermining by enlarging the end and flattening the slope. To eliminate it, a series of short jetties should be used rather than one long jetty. Of course, this cannot be done at an inlet where it is necessary to build to a certain depth of water regardless of the length, in order to create an entrance channel.

Erosion is caused by wave and current disturbance, principally during storms, and not one but both must be arrested in order to supply an adequate remedy. The author's device, a detached breakwater parallel with the shore and outshore from the low-water line, permits the current to proceed, simply reduc-

* Cons. Engr., New Jersey Board of Commerce and Navigation, Camden, N. J.

† Received by the Secretary, February 21, 1924.

ing the force of the wave sufficiently to cause it to drop its burden of sand behind the structure. In due course, as the writer takes it, the beach would be built up solid between the high-water line and the wall. Supposing the elements remained in leash a sufficient time to permit this, would the projecting section of beach keep intact in the face of an attack by a 6-knot current, such as prevails on the New Jersey coast during storms? Naturally, its life would be a function of its length, width, character of material, etc.

If connected with the shore on the weather side, thus making a combination of jetty and breakwater, it might be successful in some localities; but could not the same degree of success be realized at less expense? Assuming that the author would construct such a breakwater of heavy derrick stone for which the cost of obtaining and properly placing the materials in such an exposed location would be heavy and the maintenance expensive, probably less money, if distributed in a series of groins properly designed and suitably located over the same area, would accomplish the needed protection and extend the beach.

A sea-wall built by the Borough of Seabright* a few years ago is somewhat similar to the breakwater and quay at Ceará, Brazil, mentioned by Mr. Ripley. It is at an angle of 30° with the shore, or normal to the northeast storms, the southerly end being open. Beginning 250 ft. below the open end and 200 ft. inshore, is a rip-rap dike about 250 ft. long and parallel with the shore. Sand carried by the littoral drift has been trapped between the two structures and has filled the area between the wall and the old high-water line. The top of the sea-wall is high enough to obstruct the ocean view of those standing on the beach behind it and defeats one of the primary purposes for journeying to the coast, namely, to secure a view of the ocean. Nevertheless, it cannot be denied that the wall has been a vital protection to the municipality and the Peninsula Hotel. Again, the "hurdles" constructed by Lewis M. Haupt, M. Am. Soc. C. E., at Beach Haven, N. J., a few years ago, are somewhat similar to the author's suggestion in that they are designed to break the wave action and cause the sand to deposit behind them, thus permitting freedom of current action. This bulkhead† appears to extend in the form of a semicircle from the boardwalk, but actually one side is straight and the other curved, and the two are connected at the end by a lower row of braced sheet-piling constituting the "hurdle" over which the broken wave passes. Unquestionably, these "hurdles" have protected the boardwalk and adjacent property, but thus far have not made any permanent addition to the beach.

The Gironde River Breakwater has a base of 88.5 ft. and a maximum height of about 12 ft. and is closed at one end. Even though these dimensions could be reduced somewhat, nevertheless, the enormous cost would hardly be justified by the ratables of those political sub-divisions most in need of protection. Bathing in front of or in the neighborhood of the breakwater would be destroyed, and this is the chief attraction to those visiting shore resorts with sandy beaches.

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* See Report by the Board of Commerce and Navigation of the State of New Jersey on the Erosion and Protection of the New Jersey Beaches, 1922, p. 29.

† *Proceedings*, Am. Soc. C. E., March, 1924, pp. 369–370, Figs. 28 and 31.

# THE DISTRIBUTION OF INTENSE RAINFALL AND SOME OTHER FACTORS IN THE DESIGN OF STORM-WATER DRAINS

## Discussion*

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By Messrs. Kenneth Allen, George T. Hammond, L. L. Tribus, William T. Lyle, Adolph F. Meyer, W. W. Horner, and Allen Hazen.

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Kenneth Allen,† M. Am. Soc. C. E.—This paper brings out two noteworthy facts: (1) That there is a deplorable lack of information available regarding simultaneous intensities of rainfall at different points within limited storm areas; and (2) that what information there is, indicates that great variations occur within short distances.

The City of New York, in co-operation with its five Boroughs, is about to undertake a series of run-off experiments which, in time, it is hoped, will fill in this gap, beginning with an area of 200 acres in Brooklyn. Three Friez tipping-bucket gauges, with Friez electrical rainfall recorders, will be placed on fire-engine houses suitably located.

On August 5, 1878, there occurred a downpour in New York at a rate of 9 in. per hour, for 5 min. It was considered desirable to have a scale so open as to enable each tip of the bucket, denoting a precipitation of 0.01 in., to be readily distinguished when the rate is 10 in. per hour. To meet these requirements, the horizontal time scale adopted is 2 in. per hour, and the pencil reverses direction after recording ten tips of the bucket in a vertical travel of ½ in. The three gauge-clocks will be synchronized so as to give simultaneous records and, later, it is hoped to operate gauges in all the Boroughs in order to secure similar records over a large area.

The ability to record and interpret the highest intensities and to correlate them with the records of run-off is most desirable—a matter that has probably not been fully appreciated in some of the otherwise good work that has already been done.

In connection with the rainfall records which have been described there will be tried out a new device called an "Event Recorder." This prints automatically on a ribbon of paper the date and time of the tip of the collecting bucket of the gauge to the nearest minute. By counting the times the bucket has tipped, the precipitation can be told accurately for any period desired. It will be seen that by the use of this instrument any difficulty or error in the reading of a diagram is obviated. If it proves to be positive in action and

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* This discussion (of the paper by Frank A. Marston, M. Am. Soc. C. E., published in January, 1924, *Proceedings*, and presented at the meeting of the Sanitary Engineering Division, January 17, 1924), is printed in *Proceedings*, in order that the views expressed may be brought before all members for further discussion.

† San. Engr., Office of Chf. Engr., Board of Estimate and Apportionment, New York, N. Y.

without material lag, it will probably find a field of usefulness in other directions.

Another matter concerning which there is a dearth of reliable information is the inlet time. Few actual measurements of this appear to have been made and in applying the rational formula to sewer design it is too often the subject of mere guesswork.

When it comes to measuring the run-off itself, there are many difficulties to be encountered, and it will often be found after a careful survey of the ground that material changes in the outline of the area originally assumed as tributary to the gauging station must be made. This may be due to the connection of catch-basins or private drains with sewers which divert the flow to some other district, or to the existence of areas lying at too low a level to be drained by gravity so that ponding occurs. If a catch-basin is choked with deposit, the flow in the gutter may likewise be diverted to a different sewer. The slope of a roof, or the location of a water-shed in a park or cemetery, may divert the sewage in quite a different direction from that supposed, while the run-off may be retarded in a severe storm by inadequate inlets. In the sewer itself, there will also be a throttling of the flow, if of inadequate capacity, rendering the results of gauging worthless unless due allowance is made. The examination of the Brooklyn area in question has shown the importance of positive knowledge of all these details if the results are to be worth the cost of securing them.

In the matter of gauging there are also further sources of error. If, as customary, the flow is computed by the measurement of the surface slope, is the condition of the sewer so well known that it is safe to assume a coefficient of friction? Is there no backing up from deposits or other obstructions, or from incoming sewage below? Or is there no increase in surface slope due to the proximity of a free outlet?

There is an obvious advantage in obtaining a direct gauging of the flow if this can be done, and studies are now being made as to the practicability of doing this in a second area by a Venturi flume inserted in the sewer. If the sewer is of suitable shape and size, this would appear to offer advantages over other methods of gauging sewage, and since the matter was taken up, it has been of interest to learn that this device is being installed by Glenn D. Holmes, M. Am. Soc. C. E., Chief Engineer, in connection with the new sewage treatment works at Syracuse, N. Y.

With either method of measurement, success depends on the reliable recording of the surface level as indicated by a float. Much attention has been given to the type of recording device and to the scales of the record, and in this first district to be studied the Au Continuous water-stage recorder, modified to meet local requirements, has been adopted. In this device, a chart 10 in. wide, sufficient for a year's record, if necessary, is wound on a cylinder at a rate of 1.2 in. per hour—the time scale. A pencil on a carriage moving parallel to the axis of the cylinder records the fluctuations in water-level on a scale of 2 in. to 1 ft., while a second pencil remains stationary, making a base line for reference. On reaching the 5-ft. stage (at the top of the chart), this second pencil takes up the record, starting at the base line, while the first pencil remains sta-

tionary, marking a reference line at the top of the chart. In this way, the flow in a sewer surcharged to 10 ft. above the invert can be recorded. With the scales adopted, the correlation between the times of precipitation and run-off, as well as the variations in depth, may be determined, it is believed, with sufficient accuracy.

What it is wished to emphasize is the necessity of a thorough preliminary survey of the ground and of great care required in planning the work if the results are to be such as to command confidence.

While conducting experiments of this kind, there is the opportunity to determine, aside from the known area, the frequency of flows of less than the maximum volume, and the quantity of the dry weather flow and its variation; but the main object—determination of the maximum run-off in the sewer to be expected in a given term of years—should be kept clearly in view.

It is to be hoped that the study of rainfall intensities so ably presented by Mr. Marston will be supplemented as opportunity affords by similar studies elsewhere.

George T. Hammond,* M. Am. Soc. C. E.—The Rational Method of storm-water drainage design, which also includes the design of sewers on the combined system, marks a great advance over the various methods used in the past, and is superseding all others. The principles from which it is derived, as well as some of the various difficulties and uncertainties attending its use, have long been known to the profession. Engineers should not close their eyes to these difficulties if they expect to overcome them.

The author calls attention at the very beginning of his paper to "the paucity of data"—a point well taken. The empirical data already available have led to conclusions of great value, but many more are desirable. The greatest difficulty is that the data are incomplete in many particulars, leaving uncertainty in deductions made from them.

There is evident need for systematic study throughout the country of rainfall and storm run-off. Disregard of the nature and importance of such study, on the part of public officials and others who might help, should be earnestly combatted, and the assistance of all should be enlisted in this work.

In the early days of this study, in 1905, in a discussion† before the Society, the late Emil Kuichling, M. Am. Soc. C. E., after presenting data considered typical, and discussing the method used by him, remarked that he had made the presentation "merely to illustrate a rational mode of procedure in the storm-water drainage of urban districts, and the necessity for maximum rainfall data based on uniform instead of average intensities." He continued:

"It may also be added that while precipitation records of long duration and uniform intensity are rare, there are many such of short duration up to about 1 hour, and the tabulation of these for different localities will be of great service in municipal drainage operations, as it seldom happens that an urban district is so large and flat that the storm flow from its upper part to the final outfall will occupy more than 1 hour."

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* Cons. Engr., Brooklyn, N. Y.

† *Transactions*, Am. Soc. C. E., Vol. LIV (1905), p. 192.

It is true, no doubt, that there are many such data as those referred to by Mr. Kuichling, but, unfortunately, they are seldom available from more than one gauge, and that not supplying a perfect record. To any one who has undertaken the care of a rain-gauge for a number of years, further comment is unnecessary; and if there happen to be two gauges in company, they are seldom good friends; alas, how often one calls the other liar! How often both sleep on the job!

The speaker has been actively engaged in the design of storm-water drainage and combined storm and sanitary sewers since 1895. This period has witnessed many changing styles in design, and much real progress toward the solution of problems that in his early days seemed almost hopeless. Sometimes economic questions have been associated with these problems; sometimes there was insistent demand that cost be limited to certain figures; also, that, regardless of this limitation, an entire area must be benefited by the improvement. The only solution was progressive design, that is, providing drainage facilities, for instance, such as would take care of an area for the immediate future, and be capable of future enlargement and extension.

Within a relatively few years, the attitude of the public has changed greatly toward public works in general, and drainage projects in particular. It is now often demanded that storm drains be provided of sufficient size for any condition of development that can be foreseen; and, further, there is a tendency in some cities to provide for any possible growth or development. Among the pioneers in this study were many eminent and honored sanitary engineers, among them, the late Rudolph Hering, Charles E. Gregory, and Emil Kuichling, Members, Am. Soc. C. E. Their labors and studies, their writings and discussions, illuminate the difficult pages of many reports. More than twenty years ago, Mr. Kuichling instructed the speaker how to use the rainfall data then available in Brooklyn, in the Construction of an ordinary intensity curve, for solving local sewerage problems. This curve was developed from a continuous record produced by a rain-gauge maintained by the Water Department in Brooklyn, for a period of about fourteen years. The curve was:

Intensity (equivalent to cubic feet per second per acre), $I = \frac{150}{t + 20}$, in which,

$t$ is the time of concentration, in minutes, from the beginning of the intense period. This curve is used now, in Brooklyn, in connection with the Rational Method of design. No change has been found necessary to date, largely, perhaps, on account of the lack of more consistent data, and also because it appears to be satisfactory.

The official acceptance of the Rational Method has been much delayed in many places by what may be called ultra-conservatism. The influence of Mr. Hering's work, through his records of the gauging of the Minetta Brook area in New York City, and the widely prevalent opinion held in official quarters, that drainage designs should be based on local sewer gauging, rather than on the uncertain, and often inconsistent, results obtained from a few rain-gauges, delayed the introduction of this method in many places. Its adoption by all the Boroughs of Greater New York was largely due to the writings

of the late Mr. Gregory and of Kenneth Allen, M. Am. Soc. C. E. The speaker at all times has taken much interest in the Rational Method and advocated its use wherever sufficient data were available. He recognized years ago the correctness of the theory and regretted the lack of data which he considered necessary for its scientific application. It was with much the same view that the Chief Engineer of Sewers in Brooklyn wrote,* concerning the Rational Method, as follows:

"This method is certainly correct in theory, and is a distinct advance over other methods in use, but to be able to use it successfully we must know with reasonable accuracy, the time required for storm water falling upon the farthest points of the water-shed, to concentrate at the nearest sewer inlets. This requires also a knowledge of the rapidity of flow of storm water along street gutters, from yards, and over improved and unimproved surfaces generally. Comparatively simple observations would determine this, or, at least, would add much to our present knowledge of the subject.

"At present, however, there is little or no definite information in regard to it. It is confidently predicted, however, that in the future when experimental data have been obtained sufficient to make Kuichling's method of procedure more certain in the results obtained, it will come into more general use."

It is apparent from the data presented by the author that, although drainage areas of the size usually considered in the design of drains and combined sewers require the assumption that the rate of rainfall intensity will be uniform, areas of a larger size do not warrant the assumption of so great an intensity, if the entire area is considered. Where, however, should the dividing line between these conditions be? What extent of area shall be taken as of the first class, and what of the second? Unfortunately, it does not yet seem possible to discriminate within much more than a probability, in solving a difficult problem of drainage design, especially if an unusual type of area is involved, or its size exceeds 1 000 acres.

In forecasting the run-off of the area tributary to Jones Falls, in Baltimore,† Md., in 1906, the speaker, then Division Engineer of the Storm Water Division, could find no method of obtaining a result that would give more than a probability of the facts. The area drained was about 60 sq. miles, partly within, and partly outside the City of Baltimore; much of it was undeveloped land, almost mountainous in its topography as shown by the contour map of the U. S. Geological Survey. Every available method of computing run-off was investigated. Gauge-readings taken by the U. S. Department of Agriculture were available, but represented only one point in the area. They, however, afforded the following expression of the maximum rainfall intensity occurring in 12 years‡: $I = \frac{300}{t + 25}$.

What value should be given to the run-off coefficients and for what extent of area should it be assumed that uniform precipitation would apply, and how much should the expected run-off per acre be reduced in proportion to its extent?

* Annual Report of the Borough President of Brooklyn, 1906.

† Report of the Sewerage Commission of the City of Baltimore, 1906, pp. 66-83.

‡ *Loc. cit.*, Plate 6, Rates of Rainfall, 1894–1906, inclusive.

This problem required solution in providing a drain several miles long. taking the place of the stream. The drain, which passed north and south through the center of the city, must be large enough to prevent flooding. Not only was the run-off expected under ordinary conditions important, but also the frequency of intense rains; and there were practically no data on which to decide these important questions.

The solution of this problem had to be accomplished in other ways. No gaugings of the stream under storm run-off conditions were available; but there had been notable floods from time to time that had covered some of the streets of Baltimore. One part of the stream ran through a gorge which no flood had been known to overflow. As floods had caused damage to bridges, retaining walls, and other property, surveys had been made, covering possibly half a century, showing the extent to which the waters rose during notable floods. It was found possible from these and other data obtained from property owners along the stream, to plot the greatest known hydraulic slope of the stream in flood, and as this included parts of the stream that had not overflowed the gorge, cross-sections were obtained at intervals for a considerable distance, in which the velocity and quantity of flow per second could be computed.

For this computation, Kutter's formula for velocity was selected as the most reliable. The value of $n$ was only decided on after an extensive search of engineering and hydraulic literature.

After completing these studies, on the result of which the Jones Falls Improvement was designed, the speaker was informed that several run-off formulas gave results that checked this work, if proper coefficients were applied. But who could have supplied the correct coefficients in advance with the data then available?

It should be noted, however, that the result of this forecast was approved by the consulting engineers who reviewed the design, one of whom subsequently informed the speaker that with coefficients selected from his experience, he had obtained a result by the use of a well-known formula, giving the run-off within 5% of the forecast!

The Jones Falls Improvement, consisting of three ducts, each 20 ft. wide, has now been in service about ten years. It has had to carry a number of notable floods, but has not yet been called on to serve the maximum flood for which it was designed.

In applying the Rational Method to the design of the storm-drainage system of Baltimore in 1906, it was at first considered necessary to approximate the result, using McMath's formula, and then apply the Rational Method.*

The ordinary curve of rainfall intensity was determined to be, $I = \dfrac{105}{t + 10}$.

At that time, the data on which to base coefficients of run-off were meager. Rain-gauge observations extended over only a short period, and that for one locality only. Before the work had proceeded far, however, the Rational

* Report of the Sewerage Commission of the City of Baltimore, 1906, pp. 68–70, and Plan 6.

Method took precedence, and the McMath formula was used as a check. No data were available as to the distribution of intensity over large areas, and some of the results reached by the Rational Method appeared highly irrational and improper. As a result, sizes were modified by a coefficient which for very large areas diminished the run-off per acre.

There is obvious need of much more data regarding the habits of storms, their similarity, and their departure from the expectations of observers, especially the laws (if such there are) that govern their frequency, and the effects of topography. Outside the data presented by the author, there are few, if any, concerning the distribution of rainfall over a storm area, and, for this reason, in particular, his paper becomes a contribution of importance to science.

Of the effect of storm direction over an area on the drains, little is known and considerable is assumed. The tendency of cycles of high rainfall intensity to follow each other at rather short intervals, is also a subject of much importance, especially along the Atlantic Coast. Methods of determining the value of the various coefficients used in the formulas for obtaining the quantity to be carried by drains, would afford a fruitful subject for further study, as would also the many other matters concerning which engineers have had to piece out the lion's skin of fact with the fox's skin of assumption.

It is quite possible to overcome all the difficulties referred to, and any others which might be mentioned. It is helpful to reflect that all are being studied by some of the brightest and most capable men in the profession, and to have faith that these problems will yield to their enthusiastic efforts.

The speaker has long thought that the assumption of a uniform rate of precipitation throughout the tributary area under discussion is a fallacy if the area exceeds 1 000 acres; he has been in the habit of making a progressive reduction for such conditions, although he had no data on which to justify his conclusion.

The interesting comparisons presented by the author between New Orleans and Boston data, obviously indicate the possibility of using data obtained at widely separated points, if the frequencies of the occurrence of intensities and durations are translated—or rather interpreted—properly. This opens a wide field for investigation and further study.

It is obvious that progress is being made in rainfall intensity and run-off investigations, and that the designer of drainage works has immensely more information to work with now than he had ten years ago. Much remains to be accomplished, however, before engineers are entitled, strictly speaking, to apply the name "Rational" to this method, if by the name is meant a method thoroughly based on scientific data and tested by experiments that leave no doubt in the mind of the searcher for scientific truth.

L. L. Tribus,* M. Am. Soc. C. E.—Mr. Hammond reminded the speaker of former conditions in the Borough of Richmond, one of the other divisions of the City of New York.

* Cons. Engr. (Tribus & Massa), New York, N. Y.

Prior to 1902, although for several years it had been a part of the city, Richmond was nothing but an aggregation of small communities covering, perhaps, 10 or 12 sq. miles in a long water-front belt subject to urban conditions, supported by suburban developments, and still, further, by open country.

One of the speaker's first problems on taking official engineering control of that Borough was to provide a storm-water sewer for about 320 acres, including each of the previously named characteristics. Among other peculiar conditions was the fact that the upper tributary area was a rather high plateau, from which the drainage passed through a narrow neck or gorge, spreading out into a broad valley, only slightly raised above sea level. This valley again contracted into a long rather narrow valley largely populated.

Many years previously, the lower portion of this area had been partly served by a roofed drain, approximately 8 ft. wide and 5 ft. deep. The problem was to determine the size of sewer for this whole territory for immediate use. There were no data; the chief City officials, with control of appropriations said, "Oh, no, the population there does not require big sewers; you do not need money."

Finally, after many petitions had been received, citing loss of work and sickness, even death, due to lack of drainage, the money was appropriated.

Similarly, with rain-gauges, it was necessary to buy them first and explain it afterward. Then, by means of a weir, the flow was gauged, and between the two much information was obtained.

William T. Lyle,* Assoc. M. Am. Soc. C. E. (by letter).†—The chief merit of this excellent paper lies in its stimulating appeal for continued researches in local rainfall distribution.

It is stated that for areas up to about 500 acres the variation of rate is so small that the usual assumption of a uniform rate of rainfall is probably correct, but that for larger areas a reduction of rate may be necessary. It is reasonable to suppose that the area within which no reduction is required will be greater for storms of moderate intensity than for storms of great intensity. The isodyhyetals of Plate I‡ establish this supposition. These reductions should be considered, of course, for Gulf Coast cities and for other places with large undeveloped tributary areas.

The author's method of applying data at New Orleans, La., where storms are frequent and intense, to the Boston, Mass., District by altering the frequency of the storm, is highly scientific, and if proven to be correct should aid in the accumulation of valuable intensity-area curves for other cities also.

For a long time, the writer has been trying to find fuller data on imperviousness. It is easy to calulate the areas of pavements, roofs, sidewalks, and other impervious coverings, surfaces which will be impervious with a slight wetting, but to estimate the correct imperviousness of grassed areas is unsatisfactory. The matter of inlet time, although not important for small areas, may become highly important in the case of large outlying districts. Could not

* Prof. of Civ. Eng., Washington and Lee Univ., Lexington, Va.

† Received by the Secretary, January 11, 1924.

‡ *Proceedings*, Am. Soc. C. E., January, 1924, p. 33.

such investigations be made in the National hydraulic laboratory recommended by John R. Freeman, Past-President, Am. Soc. C. E.?*

The velocity and direction of storms, as observed by W. W. Horner, M. Am. Soc. C. E., and commented on by the author, are worthy of fuller investigation as affecting the size of storm sewers. Such studies would evidently involve many more rain-gauges than are now available. It would also appear that centralized control of rain-gauges in any locality is highly desirable, in order that they may operate synchronously and dependably.

In conclusion, the writer would observe that the time of concentration as stated in the summary† of the Rational Method may not be the time required for water to flow from the most remote point of the tributary area to each point on the drain. Exceptions to this are caused by peculiarities of shape and the topography of undeveloped tributary areas. In the case of a long narrow area, for instance, the storm-producing maximum discharge might be of lesser duration than that required for "the most remote drop to reach the point in question"; with the reduction of contributing area, there might be an overbalancing increase of rainfall intensity.

ADOLPH F. MEYER,‡ M. AM. SOC. C. E. (by letter).§—This paper presents valuable data pertaining to a phase of rainfall about which practically no specific information has heretofore been available. Although it has long been recognized that, as a general proposition, intense downpours occur over limited areas, such detailed observations as those presented by the author are needed to establish the relation between intense precipitation and storm area expressed in terms of frequency of recurrence over a specific area.

Without in any way desiring to detract from the value of the paper, the writer submits that this is not the abstract relationship between intense precipitation and storm area in terms of precipitation at the storm center, as presented by the author. In determining frequency of recurrence of given rates of intense precipitation at Chestnut Hill, the author presumably used all rates of given magnitude, which have been recorded by the Chestnut Hill gauge, without knowledge of where the storm center passed. Consider those storms in which the maximum precipitation did not occur at Chestnut Hill; in those instances—and these necessarily constitute the majority of all storms used in the frequency determination—the recorded precipitation at Chestnut Hill was less than at some adjacent point, and, therefore, the average precipitation over a given area centering at Chestnut Hill necessarily was not much less, and, in many storms, unquestionably was actually greater than that recorded by the Chestnut Hill gauge.

Using the author's data, the writer was enabled to make a preliminary study of other storms than those which centered at the Brookline Town Hall; several of these storms showed a higher average precipitation over a circular 5 000-acre area centering at this gauge than the precipitation recorded by the gauge, in accord with the statement given.

* *Proceedings*, Am. Soc. C. E., August, 1923, p. 1185.

† *Loc. cit.*, January, 1924, p. 21.

‡ Cons. Hydr. Engr., Minneapolis, Minn.

§ Received by the Secretary, January 17, 1924.

It is probably true that in those storms in which the intensity recorded at the station was very high and the storm, therefore, was of great infrequency, the maximum precipitation occurred at the gauge and the average precipitation over the adjoining area was materially less. A reduction is unquestionably warranted in those cases, but the writer doubts the advisability of applying reduction factors to storms which recur every 5 or 10 years, over areas of 5 000 acres, or less.

The writer believes that, in analyses of this kind, the relation of precipitation to area should be determined from a consideration of all storms showing average frequencies of recurrence of, say, once a year, even though the central gauge did not show the maximum recorded precipitation; and that the average precipitation should be determined over areas of given size having their center at the central gauge. For securing information of this character, the best arrangement would appear to be a central control gauge and several others located about equally distant from it and preferably not more than 1 mile away. Practically all the storm maps presented by the author can be re-drawn, adding a fictitious intermediate station with greater rainfall than that recorded by any one of the existing gauges, and still retain a rational appearing storm map. In this respect, maps of storms lasting only 1 hour or less differ radically from maps of storms lasting 1 or 2 days, where several stations usually record about the same amount of precipitation and other stations record a large percentage of the maximum. Unless three or more gauges record about equal amounts, say, within 20%, the location of the storm center is hardly known with reasonable certainty.

The second phase of the subject relates to the frequency with which given rates of intense precipitation will probably recur at a given station.

At the outset, the writer agrees that climatic conditions, such as temperature, humidity, wind, and frequency of thunder-storms; that topographic features, such as mountains, valleys, and lakes; that geographic position, such as proximity to the ocean with its varying currents; that location in the paths of cyclones, or to the windward or leeward of mountain ranges, and the like—that these factors exert a great influence on the frequency of given rates of intense precipitation.

On the other hand, he believes that large areas of the United States have sufficiently similar climatic conditions and topographic features to permit grouping all rainfall stations in those areas, for the purpose of determining average frequencies of given rates of intense precipitation. He believes that the climatic and topographic differences which remain after such a grouping has been made, are less important than the observed differences in intense precipitation at adjacent stations, and in succeeding decades of time at the same station, which do not result from definite physical causes, but solely from the fact that intense rain storms occur in an irregular, haphazard manner. A study of the published records of the Weather Bureau yields ample evidence in support of this view.

Having divided precipitation stations into groups, it goes without saying that many stations will lie near the borders of these groups and that the usual

methods of interpolation and weighted averages will be applied by the intelligent designer in arriving at his conclusions. A station located directly on the border line will necessarily share equally in the rainfall characteristics of the two adjoining groups.

If the available precipitation records for every station, with its varying climatic and topographic features, extended over at least a century or two, the frequency of given rates of precipitation would be definitely known and there would be no need for discussing this subject. Few cities, however, have continuous records of intense precipitation for even half a century. Such records usually give a fairly good indication of the frequency of ordinary downpours, but, at best, they constitute an uncertain basis for the estimation of unusual storms to be expected with a frequency of once in 10 or 20 years; and such short-term records are utterly inadequate as the basis for predicting the probable frequency of the extraordinary storms that may recur once in a generation.

Ever since the writer made his detailed study of nearly 2 000 intense rain storms in the United States,* he has held the view that the records of a number of rainfall stations in a region of somewhat similar climatic and topographic features can be combined to produce an equivalent long-term record that is a far better indication of the frequency of given rates of intense precipitation for a given city than the records of that city alone, even when those records cover a period of 50 years or more. He has stated his reasons for this conclusion in the following terms:

"Intense rainstorms usually cover only a few square miles. Observation stations 5 or 10 miles apart usually show about as much dissimilarity in the rates of excessive precipitation during intense rainstorms as stations 50 or 100 miles apart. As there are only about 200 Weather Bureau stations in the United States at which continuous records of precipitation are being secured, it is apparent that only a very few of the excessive rainstorms which actually occur, are being recorded. In a few of the larger cities, of course, municipal organizations are maintaining a number of observation stations and are thus obtaining more complete data.

"The records of adjacent observation stations well indicate the irregular manner in which excessive precipitation occurs. During the New York City storm of October 1, 1913, for example, twice as much rain fell in 2 hours at Borough Hall in the Borough of Richmond, than at the United States Weather Bureau station in the Borough of Manhattan, 5 miles away. It is safe to say that, taking the country as a whole, doubling the number of Weather Bureau observation stations would double the number of records of excessive precipitation obtained. For the same reason the records of several stations in one region are virtually equivalent to a longer record at a single station. One record supplements the other, making a combined record which is far more representative of the rates of precipitation to be expected in the given region than the records of a single station."

Statistics show that "stations with a disproportionate number of very excessive storms have usually had a deficiency of ordinary storms. Other stations that have experienced many short storms have had comparatively few long ones and *vice versa*."

* "The Elements of Hydrology," John Wiley & Sons, N. Y., 1917.

The author's Table 5* gives the frequency of given rates of intense precipitation at Chestnut Hill, based solely on records covering a period of 38 years. The first record in this table is a precipitation rate of 9.1 in. per hour for 5 min., recurring with a frequency of once in 38 years. The question is: Does this record state a probable fact?

A detailed study of 745 station-year records of precipitation in the United States indicates that only ten downpours exceeding in magnitude even 8.4 in. per hour for 5 min. have been observed at the stations covered by this study. Two of these downpours occurred at St. Louis, Mo., and one at each of the following stations: Asheville, N. C., Buffalo, N. Y., Cleveland, Ohio, Denver, Colo., Jacksonville, Fla., Kansas City, Mo., Memphis, Tenn., and Raleigh, N. C.

In the writer's judgment, a 5-min. precipitation at the rate of 9.1 in. per hour is not likely to recur at Boston once in 100 years, much less once in 38 years; he estimates the rate to be expected once in 38 years for 5 min. as 7.4 in. per hour, or 19% less than the conclusion indicated by Table 5. In other words, entire neglect of the relation of rate of precipitation to storm area for a 5-min. downpour might have resulted in an error of about 10%, whereas the difference in the rate of frequency of recurrence, as estimated by the author and by the writer, respectively, as their best judgment, might possibly involve an error of about 20 per cent.

To show that the records of a single station do not constitute a satisfactory basis for estimating the probable frequency of intense precipitation, the writer presents Table 11, comparing the Chestnut Hill and the Weather Bureau records at Boston for the 26-year period, from 1891 to 1916, as given in the paper† by Harrison P. Eddy, M. Am. Soc. C. E., entitled, "Maximum Rates of Precipitation at Boston for Various Frequencies of Occurrence."

It appears from Table 11 that the rates of precipitation recurring with 5-year frequency determined from the same 26 years of records, are from 8 to 16% lower, and average 12% lower, for the Chestnut Hill Station than for the Weather Bureau Station in Boston. The rates given for the writer's Group No. 3, which includes Boston, average 9% higher than the Weather Bureau records and 24% higher than the Chestnut Hill records. For the 26-year frequency, the Chestnut Hill records give rates which are from 12 to 23% higher, and which average 17% higher, than the Weather Bureau records. The rates given for the writer's Group No. 3 average 16% higher than the Weather Bureau records and 1% lower than the Chestnut Hill records.

The 5-year frequency rate for a 5-min. time interval determined from the 38-year record for Chestnut Hill is 10% greater than if the same frequency rate had been determined from the 26-year record; and the rate for the 30-min. interval is 6% smaller. These differences result from the irregular haphazard occurrence of excessive precipitation and not from tangible, physical causes.

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* *Proceedings*, Am. Soc. C. E., January, 1924, p. 32.

† *Journal*, Boston Soc. of Civ. Engrs., February, 1920.

TABLE 11.—COMPARATIVE PRECIPITATION RATES AT BOSTON, MASS., TO ILLUSTRATE THE IRREGULAR OCCURRENCE OF EXCESSIVE PRECIPITATION.

(Average Rates in Inches per Hour for given Time Intervals.)

| Time interval, in minutes. | From Eddy's Table 4: Weather Bureau. | From Eddy's Table 4: Chestnut Hill. | Percentage of difference. | Meyer Group No. 3. | Percentage of difference, Weather Bureau—Meyer. | Percentage of difference, Chestnut Hill—Meyer. |
|---|---|---|---|---|---|---|
| FREQUENCY OF RECURRENCE ONCE IN FIVE YEARS. | | | | | | |
| 5 | 4.90 | 4.15 | —15 | 5.28 | + 8 | +27 |
| 15 | 3.43 | 2.87 | —16 | 3.68 | + 7 | +28 |
| 30 | 2.27 | 1.98 | —13 | 2.54 | +12 | +28 |
| 60 | 1.44 | 1.33 | — 8 | 1.56 | + 8 | +17 |
| 120 | 0.81* | 0.73 | —10 | 0.88 | + 9 | +21 |
| Mean........ | .... | .... | —12 | .... | + 9 | +24 |
| FREQUENCY OF RECURRENCE ONCE IN TWENTY-SIX YEARS. | | | | | | |
| 5 | 6.70 | 7.50 | +12 | 6.99 | + 4 | — 7 |
| 15 | 4.47 | 5.50 | +23 | 5.03 | +12 | — 8 |
| 30 | 3.02 | 3.58 | +19 | 3.58 | +19 | — 0 |
| 60 | 1.75 | 2.04 | +17 | 2.26 | +29 | +11 |
| 120 | 1.12 | 1.28 | +14 | 1.29 | +15 | + 1 |
| Mean........ | .... | .... | +17 | .... | +16 | — 1 |

* Added from Weather Bureau data.

To illustrate further the irregular occurrence of intense precipitation, the writer presents Table 12, prepared from data published in his book, "The Elements of Hydrology," previously referred to.

Of the stations listed in Table 12 the first three, namely, Memphis, Tenn., Cairo, Ill., and St. Louis, Mo., are near the border line of the writer's Groups Nos. 2 and 3, and should average higher rates of precipitation than the last three, namely, Indianapolis, Ind., Cincinnati, Ohio, and Boston, which are near the center of the Group No. 3 area. Apparently, Indianapolis has had an exceedingly large number of short storms during the first period and Cairo has had a large number of long storms during the same period. During the second period, both stations have had about the same total number of storms as during the first period, based on the relative number of years in the two periods.

During the first 19 years, Memphis had only 17 storms against Cairo's 31, whereas during the next 7 years, Memphis had 15 storms against Cairo's 11. Similarly, during the first period, Indianapolis had 35 storms against Boston's 9, whereas during the second period, Indianapolis had only 14 storms against Boston's 8. The station showing the greatest increase in the number of storms is Boston, which had 8 storms in the 7 recent years, as against 9 storms in the 19 earlier years. This clearly indicates that Boston did not have its normal number of heavy downpours during the period, 1896-1914.

The writer urges once more with the utmost conviction, that the probable frequencies of recurrence of given rates of intense precipitation cannot safely be estimated from the records of a single rainfall station until these records extend over at least a century, and, even then, similar records from other rainfall stations, although a considerable distance removed, must be given careful consideration and great weight in reaching conclusions.

TABLE 12.—PRECIPITATION RATES AT VARIOUS STATIONS TO ILLUSTRATE THE IRREGULAR OCCURRENCE OF EXCESSIVE PRECIPITATION.

| Time interval, in minutes. | NUMBER OF DOWNPOURS EXCEEDING 0.4 IN. IN 5 MIN ; 0.9 IN IN. 15 MIN.; 1.2 IN. IN 30 MIN.; 1.5 IN. IN 60 MIN.; AND 1.8 IN. IN 120 MIN., AT: | | | | | | |
|---|---|---|---|---|---|---|---|
| | Memphis. | Cairo. | St. Louis. | Indianapolis. | Cincinnati. | Boston. | Total. |
| | **1896 TO 1914 (19 YEARS).** | | | | | | |
| 5 | 3 | 3 | 9 | 9 | 4 | 2 | 30 |
| 15 | 4 | 4 | 9 | 9 | 3 | 1 | 30 |
| 30 | 2 | 8 | 6 | 6 | 4 | 1 | 27 |
| 60 | 3 | 8 | 4 | 7 | 4 | 4 | 30 |
| 120 | 5 | 8 | 5 | 4 | 4 | 1 | 27 |
| Total......... | 17 | 31 | 33 | 35 | 19 | 9 | .... |
| | **1915 TO 1921 (7 YEARS).** | | | | | | |
| 5 | 5 | 2 | 6 | 4 | 3 | 3 | 23 |
| 15 | 3 | 3 | 2 | 2 | 2 | 0 | 12 |
| 30 | 1 | 3 | 4 | 2 | 2 | 1 | 13 |
| 60 | 4 | 2 | 5 | 2 | 1 | 1 | 15 |
| 120 | 2 | 1 | 5 | 2 | 1 | 3 | 14 |
| Total......... | 15 | 11 | 22 | 14 | 9 | 8 | .... |

Consider, for example, the excessive precipitation of 7.1 in. in 90 min. at Cambridge, Ohio, on July 16, 1914. Such a downpour was not equalled in any one of the 745 station-year records which the writer has studied, and, in his judgment, the probable frequency of the recurrence of such a high rate of precipitation in the same spot is less than once in 1 000 years. Out of the 745 station-year records between 1896 and 1914, referred to, the maximum recorded rainfall for 90 min. is 6.03 in. at Galveston, Tex., during the storm of April 22, 1904. During this storm, 7.58 in. of rain fell in 120 min. The use of the Cambridge record alone for that locality, without reference to prevailing rates of precipitation elsewhere, would result in a grossly inaccurate conclusion as to the frequency of given rates of precipitation. Works of improvement designed on that basis would result in an unwarranted waste of the taxpayer's money. If it is admitted that other precipitation data should be given consideration in such an instance as the Cambridge record, then is not the writer's view conceded?

W. W. HORNER,* M. AM. SOC. C. E. (by letter).†—The author is to be complimented on the work he has done on what heretofore has been considered a rather remote, if not negligible, factor in sewer design. The writer has appreciated for a long time that this unstudied factor might prove, when finally analyzed, of considerable importance. Mr. Marston's work does a great deal to clear up this subject. As he states, his data are not sufficient to furnish anything like final conclusions as to the variation of intensity of precipitation during critical storms, yet it would seem to warrant the conclusion that this variation can be neglected in designing the smaller sewer districts. In the case of long main sewers, however, draining areas of 2 000 acres or more, the reduction indicated in Table 10‡ would be sufficient to have a quite appreciable effect on the project cost.

In rolling country, the time of concentration of a 500-acre district will generally be less than 20 min., and of a 5 000-acre district, slightly less than 60 min. In the case of the 5 000-acre area and a 60-min. storm, the reduction in capacity allowed by the author's figures would be 15 per cent. Mr. Marston hesitates to recommend a reduction of run-off to this small extent, apparently because of the feeling which all engineers share that too little is known at present about the coefficient of run-off itself and that, as a result, one is not justified in using fine calculations in any detail of sewer design. The writer feels, however, that judgment as to the coefficient of run-off should stand by itself and that, for the present, it should include a factor of ignorance which eventually will be eliminated.

Having established this coefficient, engineers are justified in adopting all the refinements possible in the remaining processes of design.

The writer is glad to note that the author has stressed the three uncertainties in the use of the Rational Method. Engineers have generally considered them as two, and the third, or variation in intensity, which the author considers of minor importance (but for which he immediately supplies constructive data), was not taken seriously.

The writer would like to add emphasis to the statement that the profession is vitally interested in having additional studies made on: First, the distribution of intense rainfall; second, the coefficients of run-off from sewered areas; and, third, the so-called inlet time. In doing this, the writer confesses to a certain sense of guilt and of apology to the profession for the excellent records covering 13 years, now lying in the St. Louis office, which have been analyzed only in a fragmentary manner. It is hoped that during the coming year time can be devoted to a complete re-study of these records, and additional data presented on all three of the points mentioned.

ALLEN HAZEN,§ M. AM. SOC. C. E. (by letter).‖—The problem presented by the author is an important one. Records of the requisite degree of accuracy and of a character to make them available for its discussion, are few; even

* Chf. Engr., Sewers and Paving, St. Louis, Mo.

† Received by the Secretary, February 13, 1924.

‡ *Proceedings*, Am. Soc. C. E., January, 1924, p. 43.

§ Civ. Engr. (Hazen & Whipple), New York, N. Y.

‖ Received by the Secretary, February 23, 1924.

the excellent data presented are hardly adequate. On the other hand, however, the matter is so important as perhaps to warrant stretching the data further than would be permissible under other circumstances. In any event, the result reached must be regarded as a tentative first approximation.

The writer suggests that a distinction might well be drawn between the highest rate of precipitation found for any 1 000-acre tract, in any storm, and the corresponding rate for a particular area of the same size. Apparently, the author has taken the 1 000 acres (or other area) that could be measured from the diagrams, to show the highest rate of precipitation for each storm, as the basis for the subsequent procedure. These areas would never twice be the same.

The data are perhaps hardly sufficient to warrant the study on another and more difficult basis, but the writer feels that for particular areas, as, for instance, the areas tributary to certain sewers, it would be found that the falling off in rate of any specified frequency with increasing area would be more rapid than Mr. Marston finds from his procedure.

The author mentions an unusual precipitation at Cambridge, Ohio. Other cases of this kind come to light from time to time. They are so far above the general run of data as to suggest that they are exceptions and not extreme cases coming under the normal law of variation. The author speaks of this as being, perhaps, a 300-year storm, which is certainly a moderate estimate of its rarity.

Suppose, for example, that there were 500 observers, including volunteers in Ohio, or in some larger area, and that this record at Cambridge is the maximum recorded by any of them in a 20-year period. If this was so, the record would be a maximum for 10 000-record years. This, obviously, is not precise, as neither the number of stations that could properly be brought into comparison, nor the number of years, could be determined, but it does give a broad general check. If this record is taken as the 10 000-year term, that is, the amount such that there is a 0.0001 chance of its occurrence at any one station in any one year, and compare its magnitude with rainfalls that do recur more or less regularly, at much shorter intervals, the result is not unreasonable, as studied by probability methods. If it was not for this record, and others more or less like it, engineers might well be led to question the propriety of using these methods.

It is to be hoped that this paper will stimulate the collection of more and better data. If these records could be compared with simultaneous run-off records from actual catchment areas, their value would be still further increased.

# ANALYSIS OF THE STRESSES IN THE RING OF A CONCRETE SKEW ARCH

## Discussion*

By Messrs. Almon H. Fuller and Edward Godfrey.

Almon H. Fuller,† M. Am. Soc. C. E. (by letter).‡—This paper bears evidence of the attempt to avoid intricate and (to many engineers) abstruse theories as far as possible, in order to secure a gain in clearness, as stated by the author.

The writer feels that the author's treatment, based as it is on a clear conception of the problem and the application of a few fundamental principles of mechanics, supplemented by some of the more intricate ones, is as understandable as can be expected. He has examined this paper sufficiently to feel that a trail has been blazed, which can be followed with confidence and with fewer perplexities than any other existing route.

It seems almost too good to be true that all the stresses along the crown section may be derived from one thrust, two shears, and three moments, and that these, in turn, may be obtained by no more difficult a procedure than a somewhat tedious application of mechanics, calculus, and algebra. Clear thinking, however, is required to transform these forces into unit stresses in all parts of the crown section and in other sections. The assumption that the whole plane of any longitudinal section remains a plane needs verification in this particular instance. Stress measurements on full-sized skew arches would contribute much toward these and the other phases of the entire problem.

It is to be hoped that any existing experimental data will be contributed to the present discussion; further, that the way will yet be found to conduct the proposed tests on models. Although the author perhaps wisely refrained from lengthening the paper by including the computations for the concentrated loads on the model, it might be well to add them as a second appendix or, at least, to indicate the means of adapting to concentrated loads the expressions developed for live loads when uniformly distributed. By doing this, he might enlarge the number of readers and the general usefulness of this noteworthy paper.

Edward Godfrey,§ M. Am. Soc. C. E. (by letter).‖—Many failures have occurred with skew arches and basket-handle arches. The author illustrates

* This discussion (of the paper by J. Charles Rathbun, M. Am. Soc. C. E., published in February, 1924, *Proceedings*, and presented at the meeting of February 6, 1924), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Prof. Civ. Eng., Iowa State Coll.; Cons. Bridge Engr., Iowa State Highway Comm., Ames, Iowa.

‡ Received by the Secretary, February 4, 1924.

§ Structural Engr. (Robert W. Hunt Co.), Pittsburgh, Pa.

‖ Received by the Secretary, February 5, 1924.

his paper, in Fig. 1* and Fig. 10,† by basket-handle arches. Skew arches and basket-handle arches are frequently desirable for appearance, or to fit conditions; but, in the writer's judgment, the proper solution of either problem is in what might be termed "camouflage".

The reason that a basket-handle or semi-elliptic arch is not a proper structure for ordinary conditions is that the semi-elliptic curve of equilibrium pre-supposes active horizontal pressure of the fill on the haunches of the arch; and a soil does not exert an active horizontal pressure under ordinary conditions. At least, the pressure it may exert cannot be relied on for the stability of a structure.

If the semi-elliptic shape of the intrados of an arch is desirable for appearance, it can be maintained by a curve of short radius close to the springing of the arch. The extrados should be of long radius, or approximately parabolic, so that the mid-line of the arch ring (neglecting the "fillet" at the spring) will be near the curve of equilibrium for vertical loads only.

In any skew arch of the type illustrated by the author, the load has a tendency to take the nearest course to the abutment, the reaction tending in a direction normal to the barrel of the arch. This produces a twisting moment on the abutments as well as a horizontal twist on the arch ring. That the stresses and moments are exceedingly complex is evidenced by the author's solution.

The fact that a number of arches of this type have failed should be a warning to a designer who uses it. A mathematical solution, no matter how correct it may be in theory, is not enough to cover inherent faults in the type. The writer has observed that practically all structural failures, of which fifty might be named, could be classified in a few types of design.

In order to obviate this type of skew arch, and, therefore, avoid all the complex theory in its solution, it is only necessary to design the arch in separate parallel rings or ribs that can act as independent arches as regards main arch stresses. In the writer's judgment, this is the proper thing to do. Flexible ties (say, rods passed through pipes embedded in the intermediate ribs and fully anchored in the outside ribs) could be used to tie the skew arch together and obtain lateral stability.

In order to give a smooth or cylindrical surface on the intrados of the arch, a false ceiling could be hung under the arch, or the under side of each arch rib could be made to conform with the cylindrical surface. The upper side, or extrados, could be stepped off, the ribs being separated by felt paper or other suitable separators.

* *Proceedings*, Am. Soc. C. E., February, 1924, p. 135.
† *Loc. cit.*, p. 157.

# THE HYDRAULIC DESIGN OF THE SHAFT SPILLWAY FOR THE DAVIS BRIDGE DAM, AND HYDRAULIC TESTS ON WORKING MODELS

## Discussion*

By Messrs. H. B. Muckleston, H. J. F. Gourley, and B. F. Jakobsen.

H. B. Muckleston,† M. Am. Soc. C. E. (by letter).‡—The author's mathematical treatment of the problem of flow into a funnel is open to question. He starts with an investigation of the flow over a broad-crested, level, flat-topped weir, and applies the resulting formulas to the converging flow over the edge of a funnel. The writer questions if such a proceeding is warranted; assuming, however, that it is, the author's treatment of the flat-crested weir may be questioned.

Making the usual assumption that the kinetic energy of a flowing stream is some function of $\frac{V_1^2}{2g}$, where $V_1$ is the mean forward velocity of the stream, in feet per second, or, in other words, $V = \frac{Q}{A}$, in which $A$ is the area of cross-section, in square feet, the principle of least energy requires that in any open channel the total energy is a minimum when $\frac{A^3}{T} = \frac{Q^2}{g}$, in which $T$ is the width of the stream at the surface, in feet. Substituting $A\ V$ for $Q$, this reduces to $\frac{A}{2\ T} = \frac{V^2}{2\ g}$. If an elementary area, 1 ft. in width and $\Delta$ in depth, is considered, then $\frac{\Delta}{2} = \frac{V^2}{2\ g}$, and the elevation of the energy gradient above the bottom of the channel is 1.5 $\Delta$. This condition is now generally known as flow at the critical depth.

When water flows over a broad-crested, level, flat-topped weir, it passes over the up-stream edge at a depth depending on conditions at that point. It then flows across the weir, diminishing in depth and losing energy in the process, and flows off at the critical depth. Flow at the critical depth can exist only at the lower edge; because any other condition would involve a gain in total energy between that point and the lower edge, which is impossible. The

* This discussion (of the paper by Ford Kurtz, M. Am. Soc. C. E., published in December, 1923, *Proceedings*, and presented at the meeting of March 5, 1924) is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Cons. Engr., Vancouver, B. C., Canada.

‡ Received by the Secretary, December 14, 1923.

author has arrived (page 1962*) at the same result in finding expressions for $h$ and $u$. Writing $P$ for $2 \pi r$ in his formulas, they reduce to:

$$h = \sqrt[3]{\frac{Q^2}{P^2 g}}, \text{ and } u = \frac{Q}{P h} = \frac{Q}{A}$$

whence,

$$\frac{h^3 P^3}{P} = \frac{Q^2}{g}, \text{ or } \frac{A^3}{T} = \frac{Q^2}{g}.$$

On page 1963,* the author gives:

$$h_1 = \frac{2}{3} (k H)$$

That is, $h_1$ is the critical depth for the given quantity, which can exist only at the down-stream edge. Therefore, there can be no surface drop to be accounted for and, theoretically, $z$ must be zero. If this is true, the whole investigation into the quantities, $a$ and $b$, and the resulting equation for the parabola is based on a misconception.

However, this theoretical treatment of the critical depth is itself based on an assumption which may be very far from the truth. The kinetic energy of a flowing stream is never a simple function of $V$, that is, it cannot properly be represented by the velocity head calculated from the mean forward velocity, and it is this fact which accounts for the difference between the various values of $C$ in weir formulas and the value, 3.087, obtained by the theoretical treatment. It also accounts for most of the discrepancies between prophecy and performance.

The writer also takes exception to the author's statement that "the hydraulics of the cylindrical shaft and tunnel are, of course, very simple." He questions very much whether the extension of the Kutter formula, or any other empirical discharge formula, to velocities in the magnitude of 68 ft. per sec. is warranted by the present knowledge of the facts. The writer knows of no experiments at anything like such velocities, and he doubts whether there are any, more especially for quantities as large as 27 000 sec-ft. The calculations of the head lost in curvature and transition are, also, in the writer's opinion, based on unstable foundations. Such experiments as have been recorded were made with small pipes and comparatively low velocities. It may be justifiable to extend experimental deductions over a short range, but diameters and velocities of the order treated by the author are excessive departures from the base.

In the matter of entrained air, the writer is of the opinion that a great deal more trouble will be experienced than the author apparently anticipates. The writer has seen in action an hydraulic air compressor based on the Taylor principle, in which funnel flow was the only means provided for entraining the air. It worked.

H. J. F. Gourley,† Esq. (by letter).‡—The type of spillway, on a model of which the author has made such an interesting series of experiments, is the

* *Proceedings*, Am. Soc. C. E., December, 1923.

† With Sir Alex Binnie, Son & Deacon, London, England.

‡ Received by the Secretary, January 2, 1924.

same as that designed in 1911 and since constructed at the Taf Fechan Reservoir, South Wales.

A circular spillway had previously been constructed for the Font Reservoir at Tynemouth, England, about 18 years ago, but, in that instance, the overflowing water first fell into a basin forming a kind of stilling pond, and then over a circular weir formed in masonry. Below the crest of this weir, there was a rapid decrease in diameter to connect with the cylindrical shaft, and the water fell into a deep sump, which, with the stopping in the tunnel immediately up stream, presumably would divert the water. At times of heavy flood, this arrangement must have been hard on the lining of the shaft and tunnel in the vicinity of the sump. In any event, it could have had only a low efficiency.

The author's investigations and his conclusions are of particular interest to the writer, because he was unable to determine the behavior of the Taf Fechan Spillway and Tunnel experimentally. In the Taf Fechan Spillway, the granite crest was 66 ft. in diameter, rounded on its up-stream edge, and the bellmouth was struck with a radius in vertical axial planes of 30 ft., leading into the vertical shaft, 16 ft. in diameter. The desirability of introducing piers on the weir crest to prevent vortex motion and to produce a more uniform depth of flow, was considered; it was deemed inadvisable, however, to allow any projection above the crest that might trap floating dêbris, such as trees. In the bellmouth, as constructed, there are four granite fins, radial in plan, 9 in. thick, with a maximum projection of 18 in., which tapers off at the top just below the weir crest, and also at the bottom at the junction with the cylindrical shaft. The cross-sectional area of the channel which forms the access to the weir over about two-fifths of its length is so reduced on both sides of the median line as to insure a comparatively low and fairly uniform circumferential velocity. It is anticipated that there will be no great difference between the heads on the weir at the reservoir side and opposite the smallest section of the channel of approach.

The radius of the bend at the foot of the shaft is 15 ft., and there the diameter changes from 16 ft. in the shaft to 13 ft. in the tunnel. To have increased the radius would have added considerably to the cost of construction at this point. With regard to the weir, the writer investigated the flow over sharp-edged circular weirs ranging from 7 to 26 in. in diameter,* and for these sizes the coefficient in $Q = C\,L\,H^{1.42}$ increased from $C = 2.93$ to $C = 3.03$. The fact that the flow varied as $H^{1.42}$ was quite definitely established; subsequently, the index for sharp-edged rectangular and trapezoidal weirs† was also found to be less than 1.50.

Turning now to Table 7,‡ Experiments 2 and 7 are seen to lie well on the curve of Fig. 17.§ Using these experiments to determine the constant, $C$, and the index, $m$, in the formula, $Q = C\,L\,H^m$, it will be found that $C = 2.14$ and $m = 1.34$. That $C$ will have a larger value in the actual spillway, the writer

* *Minutes of Proceedings*, Inst. C. E., Vol. CLXXXIV (1911).

† *Loc. cit.*, Vol. CC (1914).

‡ *Proceedings*, Am. Soc. C. E., December, 1923, p. 1993.

§ *Loc. cit.*, p. 1995.

has no doubt, but the difference between 1.34 and the value, 1.50, taken by the author as the basis of comparison for the flow ratio,* is striking, and suggests that instead of this ratio being $\frac{1}{7\,776}$ it should be $\frac{1}{4\,380}$. The increase of $C$ may possibly increase the ratio to $\frac{1}{5\,500}$ but even this would call for a weir of about 40% greater diameter than is proposed, as it does not seem to be feasible to increase the head materially.

B. F. JAKOBSEN,† M. AM. SOC. C. E. (by letter).‡—The advantage of the ordinary weir spillway lies in the fact that it has a large factor of safety; since the discharge is proportional to $H^{\frac{3}{2}}$, it increases more rapidly than $H$. The disadvantage of the siphonic spillway lies in the fact that it is nearly independent of $H$; its advantage, in that it is possible to anticipate a threatened flood by beginning to lower the reservoir.

The proposed spillway type seems to have the disadvantages of the ordinary weir and the siphonic spillways combined. This is illustrated in Fig. 25;§ when $H = 9.5$ ft., the capacity is about 33 000 sec-ft.; when $H$ is increased to 14 ft., the capacity is increased only 3 per cent. In contrast to this, the weir type of spillway would discharge 80% more under the increased head.

In the writer's opinion, the critical part of the proposed spillway is not the upper, but the lower, part. The discharge capacity of the upper part is proportional to $H^{\frac{3}{2}}$; if the coefficient, $c$, has been estimated incorrectly, so that the actual flow is only 90% of the estimated flow, this error can be compensated for by a small variation of head, $H$. For example, an increase of head from 8.0 ft. to 8.6 ft. would accomplish this compensation; but if an error of 10% exists in the lower part, that is, in the shaft, due to an incorrect estimate of the various coefficients involved, a change of head of 0.6 ft. would not help matters. Moreover, the writer believes that the error of calculation is likely to be much greater for the shaft than for the upper part. The arrangement evidently should be such that the knee shown in Fig. 25 will lie well above the crest of the dam.

Great accuracy in determining the spillway capacity is not required, because the maximum flood is not known with any degree of certainty. Probability calculations can give only approximations, even when the records for high floods are fairly accurate, which is seldom the case, and ordinary guessing is likely to be still further from the truth. For earth dams, an excessive spillway capacity should be provided, because the safety of the whole structure is involved. In this case, the ordinary weir spillway seems to be ideal, because it increases its capacity rapidly with $H$.

* *Proceedings*, Am. Soc. C. E., December, 1923, pp. 1969–1970.

† Cons. Engr., San Francisco, Calif.

‡ Received by the Secretary, January 11, 1924.

§ *Proceedings*, Am. Soc. C. E., December, 1923, p. 2005.

# DISCUSSION ON

# REPORT OF SPECIAL COMMITTEE APPOINTED TO CONSIDER THE REPORT MADE TO THE NATIONAL FIRE PROTECTION ASSOCIATION ON THE FIRE HAZARD OF DOCKS, PIERS AND WHARVES*

By William G. Atwood, M. Am. Soc. C. E.

William G. Atwood,† M. Am. Soc. C. E. (by letter).‡—The writer would suggest that in the report of the Special Committee on Fire Prevention of Docks, Piers and Wharves, in Section III, Paragraph 2, third line from the bottom,§ the sentence be changed to read: "It should be of a material equivalent in fire-resisting quality to a reinforced concrete wall not less than 6 in. thick". A similar change is suggested in the wording in the fourth line of Paragraph 7, Section IV.‖

The reason for these suggestions is that the service records in the hands of the Committee on Marine Piling Investigations indicate that the maximum damage to concrete occurs at and immediately above low water, and that few structures are immune. A 6-in. wall is thin; the reinforcing should not be less than 2 in. from the face of the wall and, therefore, would be close to its center. Unless great care was taken to build water-tight forms, it would be almost impossible to cast such a wall without considerable permeability. The records show that concrete in this position is of uncertain life. Built as these walls would have to be, the chances of even a reasonable length of life are small, and the writer does not believe the Society should approve a recommendation for a form of construction of this questionable value. It is his belief that improvements in the binding materials used in concrete are possible, which would make the construction of such a wall reasonably practical from a standpoint of durability, but these improvements have not as yet been made in materials available in the market. It seems, therefore, that the qualification suggested for this clause is a desirable one.

* Presented to the Annual Meeting, January 16, 1924.

† Director, Committee on Marine Piling Investigations, National Research Council, New York, N. Y.

‡ Received by the Secretary, January 18, 1924.

§ *Proceedings*, Am. Soc. C. E., March, 1924, Society Affairs, p. 275.

‖ *Loc. cit.*, p. 278.

# MEMOIRS OF DECEASED MEMBERS

**Note.—Memoirs will be reproduced in the volumes of *Transactions*. Any information which will amplify the records as here printed, or correct any errors, should be forwarded to the Secretary prior to the final publication.**

## DUDLEY DIGGES BRITT, M. Am. Soc. C. E.*

Died April 23, 1923.

Dudley Digges Britt, the son of Exum Britt and Ellen Custine Britt, was born at Suffolk, Va., on May 1, 1872. He was of English descent, from a family long settled in Tidewater, Virginia.

Mr. Britt was graduated from Suffolk Military Academy in 1890 and like most young engineers of that period, engaged in railroad work immediately afterward. For the next ten years, he led the wandering life of the Railroad Locating and Construction Engineer, having been employed successively with the Norfolk and Western, Alabama and Gulf, Alexander and Rich Mountain, Columbia and Western, and the West Virginia Shortline Railroads.

In 1900, Mr. Britt opened an engineering office in Clarksburg, W. Va., and soon established a large practice in railway and coal development. He served as City Engineer of Clarksburg and Chief Engineer of the Fairmont and Clarksburg Traction Company for fourteen years. He gained his greatest engineering success in the design and construction of coal mining plants, together with their railroad connections and mine development. A large part of the coal and coke development of Northern West Virginia was accomplished under his direction.

Mr. Britt gradually became involved in coal mine operations, as well as engineering, and was President of the Corona Coal Company and the Freemont Coal Company. He was also a member of the firm of Britt-Horner and Craig, Coal Operators, and Director of the Hutton Coal Company, the Hepzibah Coal Company, the Philippi Gas Coal Company, the Clarksburg Industrial Coal Company, the Elk Hill Fuel Company, the Glen Elk Lumber Company, the Liberty Carbon Company, and the Louisiana Carbon Company.

Mr. Britt was a public-spirited citizen, with a keen interest in political and civic affairs. He was ready always to help in any good work and was one of the leading citizens, as well as one of the leading engineers, of the district. He did his part to make his world better and happier and will be remembered no less for his unquestioned integrity and lovable kindly personality, than for his great works in developing the mining resources of his State.

He was married on February 18, 1903, to Flora Camden Bailey, of Weston, W. Va., who survives him.

Mr. Britt was elected a Member of the American Society of Civil Engineers on April 1, 1908.

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* Memoir prepared by A. C. Dennis, M. Am. Soc. C. E.

## THOMAS ELLIS BROWN, M. Am. Soc. C. E.*

DIED AUGUST 15, 1923.

Thomas Ellis Brown was born in New York, N. Y., on February 3, 1856. He was the son of Thomas Ellis Brown and Adeline Hamilton Clinton Brown.

His early education was obtained in private schools. He was graduated from the Charlier Institute in 1872, and studied at the Columbia School of Mines from 1872 to 1874, but was forced to drop his collegiate work on account of eye trouble.

During 1874 and 1875, Mr. Brown served as a Rodman on the Atlantic and Lake Erie Railroad, in Ohio, and from January 1 to June 1, 1876, he was Assistant Engineer of the Gilbert Elevated Railway, R. H. Gilbert, Chief Engineer, following which he was with C. W. Clinton, Architect, as Engineer during the construction of the Seventh Regiment Armory and the Queen's Insurance Building, New York, N. Y. From September 1, 1877, to July, 1878, he was Chief Draftsman of the New York Loan and Improvement Company. Subsequently, he was employed by the Edge Moor Iron Company on a contract for a New York elevated railway, and, during the same year, also at the works at Edge Moor, Del.

In June, 1879, Mr. Brown was appointed Assistant Engineer of the Metropolitan Elevated Railway in New York, N. Y., under William F. Shunk, Chief Engineer. For a short time, he was engaged with the late Alfred P. Boller, M. Am. Soc. C. E., on surveys for a Harlem River Bridge and, in December, 1879, he was appointed Assistant Engineer of the Manhattan Railway Company, which position he occupied until August 1, 1881, after which he was employed as a Transitman on a survey for the East River and Connecticut Railroad, also under Mr. Shunk. During 1882, Mr. Brown served as Assistant Engineer of the New York Steam Company, and was then appointed Engineer of the Mutual Life Insurance Building, in New York, under Mr. Clinton, the Architect. This ended what may be termed his period of preliminary training for his more important engineering work.

Mr. Brown was employed by Otis Brothers and Company as Chief Engineer, on June 1, 1884, and from that time until his death, he was associated with this Company and its successor, the Otis Elevator Company, as Chief, or Consulting, Engineer. He was closely identified with the development of the hydraulic and electric elevators, which have made the modern high buildings feasible. A close associate in this work writes of Mr. Brown, as follows:

"In my opinion, he stood at the head of his profession as an all-round elevator engineer. I believe that he was responsible for the modern high and low-pressure hydraulic elevators being designed on sound engineering principles. As he never claimed to be an electrical expert, the development of the electric elevator, especially the high-speed gearless type, was the result of the combined efforts of several men, both mechanical and electrical engineers, including Mr. Brown. * * * I believe he was the first Chief Engineer of the Otis Company and had to put the designing departments on a standard engineering basis."

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* Memoir prepared by Otis E. Hovey, M. Am. Soc. C. E.

The notable elevator installations with which Mr. Brown was connected are too numerous to mention. The inclined elevators of the Eiffel Tower, in Paris, France, were designed, built, and installed under his direction, in 1888 and 1889. The elevators in most of the tall modern buildings in the cities of the United States were built by the Otis Elevator Company, and he participated either generally or specifically in their design and construction. The elevators in the Woolworth Tower, in New York, including the air safety cushions, built in 1914, are notable examples of his skillful and ingenious work.

During 1891, Mr. Brown became Consulting Engineer of the Otis Company and in this relationship was able to undertake many important commissions for engineering work independently of the Company. Here, again, his work was varied and included a wide range of problems. Some of the important works which he designed and which were built under his direction were the Weehawken Viaduct, of the North Hudson Company Railway, with elevators, in 1892; the Catskill Mountain Incline, finished the same year; the Glasgow Harbor Tunnel elevators and the Prospect Mountain Inclines, completed in 1895; and the Weehawken and Beacon Mountain Inclines, in 1902. The elevators of the Elbe Tunnel in Hamburg, Germany, were designed and built under his direction; as well as the numerous elevators of the London Underground Railways, which were built during 1905 and 1906.

As early as 1895, he became interested in the design of movable bridges and, in 1896, proposed an ingenious type of construction for a proposed double-leaf counterbalanced bascule bridge over Newtown Creek, Brooklyn, N. Y., for which he was awarded a prize of $5 000. During 1908, the Ohio Street bascule bridge, in Buffalo, N. Y., was built from his design, which was a development from that of 1896 for Newtown Creek. In 1914, the Hamburg Turnpike Bridge, in Buffalo, was completed from similar plans, and, in 1922, the Mystic River bascule bridge, at Mystic, Conn., was finished. This design involved improvements on his earlier plans amounting to practically a new type. This was characteristic of his work. He always sought to improve engineering practice and was not satisfied to rest with a particular type of construction, however satisfactory it had seemed at the time.

Mr. Brown's attention was not given exclusively to elevator and bridge engineering, as may be noted from his accepting a position as Chief Engineer of the Highland Valley Power Company, in Idaho, about 1903-04.

During the construction of the emergency dams on the Panama Canal, in 1911-13, his advice and assistance in perfecting the details of the hoisting machinery for the track girders and rolling-gates, was of much value.

His contributions to engineering literature were not numerous, but his paper on "Passenger Elevators"* which was presented before the International Engineering Congress in St. Louis, Mo., in 1904, was very complete and valuable. He was also joint author, with George H. Blakeley, M. Am. Soc. C. E., of a paper entitled "The Weehawken Elevators and Viaduct."† Both these papers have been published by the Society.

* *Transactions*, Am. Soc. C. E., Vol. LIV (1904), Part B, p. 133.

† *Transactions*, Am. Soc. C. E., Vol. XXVII (1892), p. 1.

He was married on May 26, 1887, to Florence Coralie Bleecker, of New York, and is survived by his widow, four sons, Clinton Bleecker Brown, of New York; Thomas Ellis Brown, Jr., of Yonkers, N. Y.; Bache Hamilton Brown, of Morristown, N. J.; and Otis de Raasloff Brown, of Newark, Ohio, four grandsons, and five granddaughters.

An intimate personal friend, also an engineer, writes of Mr. Brown, as follows:

"Looking back over the long years of my friendship with the late Mr. Thomas E. Brown, my impressions of him easily fall into four groups: his personality, his charm, his strong mentality, and his modesty.

"His personality impressed itself upon me at our first meeting at Edge Moor, 45 years ago. He seemed to radiate individuality, there was a very attractive timbre in his voice, he carried himself in a way that suggested a touch of the Cavalier: there was a finish about him which differentiated him from the rest of us.

"It was not long before I, like everybody else, fell under the influence of his charm, of which he himself was perfectly unconscious. This gift, coupled with a real goodness of heart, attracted to him a host of friends. He was 'Tommy' to the elder men, the heads of the professions under whom he served, or with whom he became intimate. They, too, felt his charm, but with their affection for him, they had a genuine respect for his capacity as an engineer.

"Brown had a very unusual mental make-up. Gifted with a strong retentive brain and an exceptional power of concentration, he never studied a subject but that he mastered it, and that knowledge was ever afterwards at his immediate disposal. The thorough grasp he had of his profession, together with his charm and personality, made him an ideal executive. He had a trait, which is not as common as it should be, the humanity of his intercourse with his subordinates. He was always ready to discuss their problems with them, and give them the benefit of his knowledge and experience. Nor did he absorb all the glory for a piece of work well done, but gave due credit to those who had assisted him. And to the end, he was always ready to give a helping hand to a fellow practitioner. Despite his friendliness, no one ever presumed on his good nature; there was a certain dignity about him which only his intimates dared to penetrate.

"Withal that Brown was a master in his chosen line, and was justly proud of his work, he was singularly modest, and allowed his work to speak for him. Take him all in all, he was one of the ablest and most lovable men it has been my privilege to know, and know intimately."

Mr. Brown was elected a Junior of the American Society of Civil Engineers on November 3, 1880 and a Member on April 2, 1884.

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## WILLIAM BARNARD FULLER, M. Am. Soc. C. E.*

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DIED JUNE 17, 1923.

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William Barnard Fuller was born at Swampscott, Mass., on November 12, 1862, and received his engineering education at the Massachusetts Institute of Technology from which he was graduated in 1883 with the degree of Bachelor of Science. Subsequently, he received the degree of Doctor of Engineering from the same institution.

* Memoir prepared by George C. Robertson, Esq., Buenos Aires, Argentine Republic.

Mr. Fuller began his professional career as Assistant Engineer of Track, Bridges, and Buildings with the Northern Pacific Railway Company, which position he held from July, 1883, to May, 1887. During this time, he also designed a water-works system for Bismarck, N. Dak., and superintended a part of its construction.

In 1886 and 1887, Mr. Fuller served as Assistant Engineer for the City of Duluth, Minn. In the latter year, he was elected City Engineer which position he held until 1891, a period covering the years of Duluth's most intensive growth.

Mr. Fuller later devoted most of his energies to the design and construction of water treatment plants. He served as Designing Engineer for the filter plants at Albany, N. Y., Utica, N. Y., Trenton, N. J., and other cities, and designed the sewage disposal plants for Columbus, Ohio, Mount Kisco, N. Y., and other places. He was also engaged as one of the Designing Engineers of the Metropolitan Sewerage System of Massachusetts and for the Passaic Valley Trunk Sewer of New Jersey, and had charge of the construction of the Boontown Dam and Tunnel of the New Jersey Water-Works Company, which furnishes the water supply of Jersey City, N. J., and various surburbs.

During six years of the various Mexican revolutions, he served as First Chief Engineer and, later, as General Manager on the construction of the dam of Santa Rosalia, State of Chihuahua, Mexico, a hydro-electric project furnishing power to the mines in the vicinity of Parral about 70 km. distant. Mr. Fuller carried this work through to a successful conclusion under the most trying conditions, having been three times a prisoner of the famous Mexican bandit, Pancho Villa, and nearly losing his life at his hands.

Mr. Fuller was well-known as an investigator of the action of concrete under various conditions, and was the author of the well known Fuller proportion curve. He had also collaborated with several authors in the preparation of textbooks on the use of concrete, and previous to the World War was Junior Member of the firm of Johnson and Fuller, Consulting Architects, of New York, N. Y.

During the World War, Mr. Fuller was engaged in the construction and design of water supply and sewage disposal plants for Army encampments in the Big Bend Section of Texas. Shortly thereafter, he left for Montevideo, Uruguay, where he was employed as Manager for the Uruguayan Branch of Messrs. George E. Nolan, Incorporated of New York, N. Y., and was engaged in the design and construction of water-works systems and roads until shortly before his death in Eagle Rock, Calif., on June 17, 1923.

Mr. Fuller was elected a Junior of the American Society of Civil Engineers on June 3, 1885, and a Member on May 1, 1895.

## RUDOLPH HERING, M. Am. Soc. C. E.*

Died May 30, 1923.

Rudolph Hering, born in Philadelphia, Pa., on February 26, 1847, was a son of Dr. Constantine Hering and Therese Buchheim Hering. Dr. Hering

* Memoir prepared by George W. Fuller, T. J. McMinn, and John C. Trautwine, Jr., Members, Am. Soc. C. E., and Charles Whiting Baker, Esq.

was a prominent physician and a leader among German-trained medical men who founded Homœpathy in America. In 1860, Rudolph Hering was sent to Dresden, Germany, where he attended the Public High School and, later, the Royal Saxon Polytechnic Institute, from which he was graduated in 1867 as a Civil Engineer.

Returning to the United States, he was employed on the surveys for Prospect Park, in Brooklyn, N. Y., under the late C. C. Martin, M. Am. Soc. C. E., and was one of a group of young engineers who, later, achieved distinction elsewhere in engineering work. Next, he assisted in laying out the extensions of Fairmount Park, in Philadelphia.

As a result of his training abroad, Mr. Hering was familiar with the use of the plane-table and had a working knowledge of graphic statics and an appreciation of the artistic side of engineering.

The year 1872 was spent as Astronomical Observer with the expedition which the United States Government sent to explore and record various features of the newly established Yellowstone Park. On his return, Mr. Hering was for three years in Philadelphia on the design and supervision of construction of the Girard Avenue Bridge. He had specialized in Bridge Engineering in college, and this work was in accord with his desires and ambitions. From 1876 to 1880, he was Assistant City Engineer of Philadelphia, in charge of bridges and sewers; this latter contact changed the course of his life work.

At that time, numerous cities of the United States suffered at intervals from serious epidemics of yellow fever. The National Board of Health was empowered to investigate the causes of these epidemics and to devise remedies therefor. With few exceptions, sewerage works in the United States at that time were meager and faulty, and far below European standards, and Mr. Hering was commissioned by the National Board of Health to make a thorough investigation of European sewerage practice. After studying conditions in large representative cities of the principal European countries, he made, in 1881, what is still the most important report that has ever appeared in America as to the fundamental principles of Sewerage Engineering.

Following his return from abroad, Mr. Hering began his consulting practice. From 1882 to 1888, his time was devoted very largely to three important engagements: the first was the supervision, in the field, under the late Gen. William L. Ludlow, Corps of Engineers, U. S. A., M. Am. Soc. C. E., of an exhaustive study of new sources of water supply for Philadelphia; the second was the service, from 1885 to 1887, as Chief Engineer of the Drainage and Water Supply Commission of Chicago, Ill., which was created to devise a method for keeping the sewage of the city out of Lake Michigan, then as now used as the source of Chicago's water supply. These investigations were the basis for the establishment of the Chicago Drainage Canal.

After leaving Chicago, Mr. Hering established himself in New York, N. Y., where he was actively engaged in private consulting practice for more than thirty years. Here, his third large investigation was conducted, that of reporting on the betterments to the basic design for the local system of collect-

ing sewers, and on the arrangements of the outfall sewers, which, on his advice, were gradually extended from the bulkhead to the pierhead lines in the old City of New York.

At this period, the germ theory of disease was beginning to receive substantial recognition from members of the Medical Profession, and Mr. Hering, having quickly grasped its significance, informed himself as to the steps necessary to the adjustment of Sanitary Engineering to this new order of things. In this task, he proceeded in a thorough-going fashion, carefully ascertaining the results of the more important investigations that were then being conducted in America and in Europe.

Although the structural aspects of engineering had been of principal interest to him during, perhaps, the first ten years of his active work, he had the vision to see that, in the rapidly changing arts of water supply and sewage disposal, it was important to keep abreast of the times, not only with regard to the earlier recognized phases of engineering, but also in the allied subjects of Biology and Chemistry.

Mr. Hering was one of the first to recommend mechanical filters for purifying water supplies at Atlanta, Ga., and elsewhere, and was connected with important water supply investigations for New York, Philadelphia, Washington, D. C., New Orleans, La., Columbus, Ohio, Montreal, Que., and Toronto, Ont., Canada, Minneapolis, Minn., and elsewhere. He was a member of the Burr-Hering-Freeman Commission that reported in 1903 on an additional water supply for New York, which report was the basis for the Catskill Aqueduct project. However, it was in the field of sewerage and sewage disposal that his activities were greatest. It was along that line, also, that his accomplishments led to his designation, many years ago, as the "Dean of Sanitary Engineering" in America. Recognition of such standing was perhaps first made by the late President Harrison, who, in 1889, appointed Mr. Hering as Chairman of a Commission including the late Frederic P. Stearns, Past-President, Am. Soc. C. E., and Samuel M. Gray, M. Am. Soc. C. E., to prepare a program for sewerage improvements for Washington, D. C.

In the field of sewage disposal, Mr. Hering was identified with a majority of the larger undertakings in America for a period of more than 30 years. Besides those already mentioned, these include projects for Providence, R. I., the Passaic Valley in New Jersey, Baltimore, Md., Reading, Pa., Columbus, Atlanta, New Orleans, Los Angeles, Calif., and many smaller cities. He was the first to introduce in America the Imhoff type of septic tank. Mr. Hering was Engineer for the sewerage system of the City of Mexico, and in association with the late E. A. Fuertes, M. Am. Soc. C. E., and J. H. Fuertes, M. Am. Soc. C. E., he designed a complete system of sewerage and a public water-works for the City of Santos, Brazil.

Examination of his professional papers by T. J. McMinn, M. Am. Soc. C. E., shows that he had made engineering reports for more than 250 cities and towns on a wide range of engineering subjects. Lists of these places and

subjects, and also of his technical writings, are filed in the office of the Secretary.

Mr. Hering was a liberal contributor to numerous technical publications, including the *Transactions* of the Society, his earliest paper, entitled "Brick Arches for Large Sewers,"* which attracted much favorable attention, having been published in 1878.

He became deeply interested in the formula of Ganguillet and Kutter, for determining the velocity of water flowing in open channels and pipes, and was instrumental in securing a publication of that formula at the Philadelphia Centennial Exposition of 1876. He contributed the most useful chapters of a book expounding this formula, entitled "The Flow of Water in Open Channels," written in collaboration with John C. Trautwine, Jr., M. Am. Soc. C. E.

Mr. Hering contributed liberally of his time to the advancement of knowledge in all branches of engineering sanitation. For many years, he was an active worker on the committees of various professional and civic movements where he considered the advice of an engineer an important element in formulating a program. His most important committee work was undoubtedly that undertaken for the American Public Health Association in the matter of the collection and disposal of refuse. He gathered statistics as to the results of operation and otherwise elucidated the practice in America and in Europe. About twenty-five years ago, he gave liberally of his time and money toward gathering information on this subject, although his activities in the field of water supply and sewerage did not permit him to publish promptly and fully the results of his investigations in the disposal of solid wastes of municipalities.

Mr. Hering was in partnership with George W. Fuller, M. Am. Soc. C. E., from 1901 to 1911, and with John H. Gregory, M. Am. Soc. C. E., from July 1, 1911 to May 1, 1917. After the latter date, his activities were confined largely to the writing of a book entitled "Collection and Disposal of Refuse", of which he was a joint author with Samuel A. Greeley, M. Am. Soc. C. E.

He never became so busy that he could not find time to help young men, many of whom came to him for advice and encouragement. This kindly interest was a lovable trait which he revealed to many, both in America and abroad, and whereby he proved a strong inspiration to many struggling young engineers.

Mr. Hering had an unusual gift of learning the new developments in European practice, and of stating them in a manner that made their usefulness available to his co-workers in America. By many, he was considered the foremost of American engineers in teaching his associates on this side of the ocean how to be wisely guided by developments abroad.

In 1907, he received an honorary degree of Doctor of Science from the University of Pennsylvania, and, in 1922, an honorary degree of Doctor of Engineering from the Polytechnic Institute of Dresden.

* *Transactions*, Am. Soc. C. E., Vol. VII (1878), pp. 252, 260.

Mr. Hering was a member of many engineering and scientific societies both in America and in Europe, among which are the following: American Society of Mechanical Engineers; Institution of Civil Engineers of Great Britain; American Institute of Consulting Engineers; Canadian Society of Civil Engineers, now the Engineering Institute of Canada, having served as a member of its Council in 1900-01; American Public Health Association, of which he was Vice-President in 1901 and President in 1913; Engineers Club of Philadelphia, having served as President in 1882; Engineers Club of Trenton, N. J.; Boston Society of Civil Engineers; American Water Works Association, and the New England Water Works Association, in both of which he held Honorary Membership; Western Society of Engineers; Franklin Institute; Verein Deutscher Ingenieure; New Jersey Sanitary Association; American Forestry Association; State Board of Health of Pennsylvania in 1883; American Association for the Advancement of Science; American Society for Testing Materials; American Society for Municipal Improvements; and a Fellow of the American Academy of Arts and Sciences.

He also held membership in The Century Association, of New York; Chicago Literary Club; Century Club, of Philadelphia; Twilight Club, of New York; Pennsylvania Society; Alliance Française, and the Outlook Club, of Montclair, N. J.; Municipal Art Association; and the Deutscher Verein, of Montclair, N. J.

At its meeting in Chicago, Ill., on July 10, 1923, the Board of Direction of the Society unanimously adopted the following resolution:

"*Whereas,* Death has removed from the ranks of the American Society of Civil Engineers one of its most honored members, a Past-Director and Vice-President, Rudolph Hering, one of the foremost pioneers in the field of Sanitary Engineering, who contributed largely to the development of the science of Sanitary Engineering in this country and to the elevation of this branch of Civil Engineering:

"*Therefore Be It Resolved,* That in the demise of Mr. Hering the Society has lost one of its most able members and the Profession a most prominent worker;

"*Be It Further Resolved,* That a copy of this resolution be spread upon the Minutes of the Board of Direction of the American Society of Civil Engineers and a copy forwarded to the family of the deceased."

The Council of the Institute of Consulting Engineers adopted unanimously a resolution, as follows:

"*Whereas,* Rudolph Hering, a charter member of the American Institute of Consulting Engineers, died on the 30th day of May, 1923, in the 77th year of his age; and

"*Whereas,* Dr. Hering was for more than 40 years one of the leading practitioners of engineering through his active connections with a large number of prominent water supply and sewerage projects in this country and was long recognized as the Dean of Sanitary Engineering in America; and

"*Whereas,* Dr. Hering was one of the foremost students of European practice in the field of Sanitary Engineering and with rare skill brought the benefits of foreign practice to this country; and

*"Whereas,* Dr. Hering possessed a friendly helpful personality which endeared him to the hearts of an unusually large number of young engineers whom he greatly inspired with his helpful advice;

*"Now Be It Resolved,* That the American Institute of Consulting Engineers mourns the loss of one of its most accomplished and valued members, and records its profound sympathy for his family in their great bereavement; and

*"Be It Further Resolved,* That a copy of these resolutions be sent to Dr. Hering's family."

The Sanitary Engineering Division of the Society has established a gold medal, to be known as the "Rudolph Hering Medal", which is to be awarded to the best contributions to the *Transactions* of the Society on the fundamentals of Sanitary Engineering.

Mr. Hering is survived by his widow, Mrs. Hermine Buchheim Hering, and five children, Oswald C., Ardo, Dorothea, Paul, and Margaret, the oldest of whom is a practicing architect in New York.

Mr. Hering was elected a Member of the American Society of Civil Engineers on January 5, 1876, and served as a Director in 1891, 1897, 1898, and 1899, and as Vice-President in 1900 and 1901.

## THOMAS HENRY McCANN, M. Am. Soc. C. E.*

DIED OCTOBER 25, 1922.

Thomas Henry McCann was born in Belfast, Ireland, on June 2, 1846, and was brought to America in 1849, by his parents who settled in New Haven, Conn. His mother died in 1853, and, in 1855, his father married a second time and brought his family to Brooklyn, N. Y.

Life at home seems to have been uncongenial to the boy, and so with the courage and self-reliant spirit which distinguished him through life, and might be inferred as inherent from the place of his birth, he ran away at the age of twelve, and did not return until after the Civil War, in 1866.

On November 19, 1861, at fifteen years of age, he enlisted and was mustered into service as a Private in Company C, 90th New York Volunteers, to serve three years. He re-enlisted in the same Company as a Veteran Volunteer, on February 20, 1864, was promoted to be Sergeant on February 10, 1865, and was mustered out with the Company and honorably discharged from the service at Savannah, Ga., on February 9, 1866.

As a child, Mr. McCann had been christened Henry, after his father. When he enlisted, for fear his father should discover his whereabouts and procure his rejection by reason of being under age, he gave the name of Thomas H. McCann, his grandfather's name. For the remainder of his life, he was known by that name.

Although attached to a command which saw more than the average of hard fighting between 1861 and 1865, in all parts of the enormous field of the Civil War, from Virginia to beyond the Mississippi, and although endowed with a naturally intelligent mind and with more than the usual powers of observation,

* Memoir prepared by Edlow W. Harrison, M. Am. Soc. C. E.

Mr. McCann's innate modesty never permitted him to capitalize his long war experience nor to use it as an asset in his future professional life. Very few of his friends in after life were aware of his youthful experience, and those who were, had great difficulty in drawing from him anecdotes and incidents of scenes and events which live forever in history, "part of which he was". His command bore a most strenuous part in the Battle of Cedar Creek in the Shenandoah Valley.

In the course of a discussion of the topography of the Appalachian Belt between the Hudson Valley and the North Carolina highlands, while waiting with the writer to be called as experts on opposite sides in a tedious case, Mr. McCann, in illustrating a point as to the influence of the mountain ranges on stream flow to the Atlantic and the Mississippi, took out a pencil and made a rough sketch of the Shenandoah Valley from Harper's Ferry down, with the gaps in the Blue Ridge which figured so largely in the strategy of the Civil War. The local knowledge he displayed was the first intimation the writer had as to his war experience in that region, and brought out a vivid story from an eye-witness of nearly half a century before, of one of the most dramatic events in all war history, which has been embalmed in poetry, picture, and sculpture for all time—Sheridan's Ride—and the turning of a disastrous defeat into a great victory, all within one day, October 19, 1864.

Mr. McCann's command had been fighting a rear-guard action from early morning, and had fallen back before the enemy for some distance along the line of the Valley Pike. A little before noon, there was a lull along their front, and the exhausted men had thrown themselves down along a roadside ditch, when the faint sound of cheering was heard up the road. The sound grew louder, and in a cloud of dust, General Sheridan, followed by a small escort, passed, waving his hat and shouting "Turn back, boys! we'll whip them yet. Turn back!" As Mr. McCann described it, the effect was electrical. The tired men sprang to their feet, waved their caps and cheered. Companies were hurriedly reformed, and down the road and across the fields the shouting men followed the General toward the enemy.

The first idea of future engineering work came to Mr. McCann during the siege of Port Hudson in June, 1863. He had been detailed to assist an officer on a mortar battery. His Commander seems to have taken a fancy to the boy, and they talked much together. With the thirst for knowledge which Mr. McCann always exhibited, this intercourse of about six weeks, with constant experience in the study and practice of explosives and projectiles, with a young man fresh from the school-rooms of West Point, had a great effect, and he then determined that after the war he would make engineering his profession. On his discharge in 1866, a few months before his twentieth birthday, he returned to New York, N. Y., and succeeded in obtaining a position as Rodman in the Sewer Department of the Board of Public Works of the city, under the late Albert Wingate Craven, Past-President, Am. Soc. C. E., and one of the founders of the Society. At the same time, he attended a city night school and afterward the Cooper Union night school, from which he was graduated in 1871.

Mr. McCann was a typical example, in the last generation, of a group of pioneer American engineers, largely self-taught, and mainly in the school of hard experience, not specialists, but, of necessity, all-round men who laid the foundations and reared the scaffolding of the greatest miracle of physical construction ever known in the world's history. In the limited field in which his professional practice lay, mainly Hoboken, Hudson County, and Northern New Jersey, his work covered operations which, in value and importance, compare favorably with the average of engineering work of the country.

In 1872, Mr. McCann removed to Hoboken and was engaged as an Assistant in the office of Messrs. Spielmann and Brush (the late Arthur Spielmann and Charles B. Brush, Members, Am. Soc. C. E.). He remained with that firm and, after Mr. Spielmann's death, with Mr. Brush, until 1889.

In that year, Mr. McCann formed a partnership with Mr. Albert Beyer, under the firm name of Beyer and McCann. During these engagements as Principal and responsible Assistant of those firms, he worked continuously in the important developments in Hoboken and North Hudson County, which have converted this area of small, detached suburban settlements, containing several miles of unimproved water-front on the Hudson River, into a thickly inhabited community, with great industrial plants and railroad terminals, and miles of improved water-front utilized by a large proportion of the foreign commerce of the Port of New York, including many of the larger vessels of the world's tonnage.

In the fifty years of Mr. McCann's professional life, he had engineering charge of the construction of steamship piers to accommodate the 4 000 or 5 000-ton vessels of the great German lines in Hoboken, and, as the years passed, the reconstruction and extensions made necessary by the advent of larger vessels, until, before his death, he had been engaged as Expert Consultant for the steel and concrete deck piers for the *Leviathan*.

As Assistant to Mr. Brush, in 1884, Mr. McCann was Resident Engineer in charge of the construction of the plant of the Hackensack Water Company from New Milford, N. J., to Hoboken, including several large reservoirs, a water tower on the Palisades, and a distribution system covering many miles, including Englewood, Hackensack, and other towns in Bergen County, New Jersey, and all the North Hudson territory, including Hoboken.

Mr. Brush was Engineer of the Hoboken Land and Improvement Company, which owned the holdings of the Stevens family estates, including the Hoboken Ferries. In addition to work on the piers, Mr. McCann reconstructed the slips and ferry-houses of both upper and lower ferries in Hoboken, and the Barclay and Fourteenth Street ferries in New York. He was also Chief Assistant for the unfortunate enterprise of English promoters to tunnel the Hudson, which ended in the flooding of the bore and the death of the men at work. After twenty years under water, the old work now forms part of the up-town tube of the Hudson and Manhattan Railroad Company. Mr. McCann was probably the first engineer to give lines and grades below the bottom of the Hudson. He also designed and constructed the first elevated trolley line in New Jersey, from Hoboken Ferry to the Heights, as well as the two inclined planes with platforms, now used for vehicular traffic.

In 1893-95, he designed and constructed the so-called "White Line Trolley" from Hoboken to Passaic, N. J., now part of the Public Service System, with the power-houses, car barns, and bridges, complete.

For some years, he was associated with and assisted Gustav Lindenthal, M. Am. Soc. C. E., in consultation on local details, borings, etc., for the proposed Hudson River Bridge, from Twenty-third Street, New York, to Twelfth Street, Hoboken.

In 1893-95, he designed and built the bridges over the Hackensack River and Overpeck Creek on the Hackensack Plank Road; the bridge over the Passaic River at Jackson Street, Newark, N. J.; and the long draw-bridge on the shore highway over the Raritan River at Perth Amboy, N. J.

For several years, Mr. McCann served as Chief Engineer of the Shore Railroad in Hoboken, which has been a short, but important, link in tying up the great railroad terminals with the steamship docks.

In addition to these specific works, Mr. McCann was constantly engaged, for fifty years, in the construction of important municipal works, paving, park drives, boulevards, sewers, pipe systems, borings and soundings, and many heavy retaining walls.

He was an omnivorous reader of scientific works, and remembered all that he read. He was also a careful, intelligent, and trusted expert in litigation on engineering matters. He found time, late in life, to write a short history of a private soldier in the Civil War, especially interesting as to Sheridan's great campaign in the Shenandoah Valley in 1864. In 1890, he translated, with comments, a German report on the improvement of the Weser River from Helgoland to Bremen.

While President of the Board of Education of the City of Hoboken, Mr. McCann made a voluntary study of modern school construction in comparison with the old-fashioned structures then existing there, with the result that his ideas and plans were adopted and incorporated in the first modern school building erected in that city, and which has since been followed by others.

He died on October 25, 1922, at his home in Hoboken, where he had lived as a widower for several years.

Mr. McCann was elected a Member of the American Society of Civil Engineers on April 2, 1890.

## WILLIAM BAIRD PATTON, M. Am. Soc. C. E.*

DIED NOVEMBER 29, 1923.

William Baird Patton, the son of William Wagner and Anna Elizabeth (Baird) Patton, was born October 14, 1860, in Germantown, Pa. His father was a veteran of the Civil War. His mother was of Revolutionary descent, being a grand-daughter of Dr. Absolom Baird, a surgeon in the Continental Army, and a great grand-daughter of Major John Baird, who served under General Braddock.

* Memoir prepared by Lyonel Ayres, M. Am. Soc. C. E.

Mr. Patton was educated in the public schools of Philadelphia. He entered the University of Pennsylvania in 1876 and was graduated in 1880 in the Civil Engineering Course, with the degree of Bachelor of Science.

He was in the City Survey Department of Philadelphia until 1881, when he went to Duluth, Minn., in the employ of the Duluth and Winnipeg Railroad as Locating Engineer, remaining in that service until April, 1883, when financial difficulties caused the Railroad Company to cease all operations.

He was appointed Engineer of the Village of Duluth and served three years as such. In 1888, he was elected County Surveyor of St. Louis County, Minnesota, and re-elected in 1890 and 1892.

In April, 1895, Mr. Patton was appointed City Engineer of the City of Duluth, serving three terms, until 1897. He was re-appointed in 1901 and served for three years. During all this time, he was also engaged in private practice as a Surveyor and Engineer. While City Engineer, he designed the Municipal Water-Works plant, and started its construction. He also designed and supervised a number of other important municipal improvements, not only in Duluth, but in many of the villages and cities in the northern part of Minnesota.

In 1899, he designed and supervised the construction of a dry dock at Superior, Wis., which at that time was the largest dry dock in the world on fresh water. He also designed a concrete dry dock for Texas City, Tex., but this was never constructed.

In 1903, with Thomas F. McGilvray, M. Am. Soc. C. E., he organized the Duluth Engineering Company to carry on a general civil engineering and surveying business, serving as Manager thereof until his death. On October 11, 1920, with Messrs. Samuel S. Gannett, of Washington, D. C., and John G. D. Mack, of Madison, Wis., Mr. Patton was appointed by the United States Supreme Court to determine and fix the boundary line between Minnesota and Wisconsin on the St. Louis River. The work was carried on during the winter of 1920 and 1921, in order to take advantage of surveying over the ice.

In February, 1917, Mr. Patton passed the Government examination for rank of Major in the Officers' Reserve Corps, Class B, in which he served until January, 1918, at which time ill health necessitated his retirement from the service. At the time of his retirement, he was stationed at Camp Lee, Virginia.

Mr. Patton was an enthusiastic Mason and was revered by all members of the fraternity with whom he came in contact. He served as Master of Palestine Lodge, A. F. and A. M., at Duluth, for four consecutive years; three years as High Priest of Keystone Chapter, R. A. M., and one year as Commander of Duluth Commandery, K. T. He was honored in 1910 by being elected Grand Master of the Grand Lodge of A. F. and A. M. of Minnesota, and was Grand Patron of the Order of Eastern Star. He was a Thirty-third Degree member of the Scottish Rite Branch of Masonry.

He was an Honorary Member of the Lodge of King Solomon's Temple of the Grand Lodge of England, having been one of four Masons in the United

States to receive that honor, the others being Presidents Theodore Roosevelt and William Howard Taft, and General T. J. Shryock, of Maryland.

Mr. Patton had been a member of the Duluth Chamber of Commerce from its organization and had served as Chairman on some of its most important committees.

He was a member of the Baptist Church and active in its work, particularly in the Sunday School, having been Superintendent at one time for sixteen consecutive years.

Mr. Patton was a man of unusual character and integrity, unsullied reputation, and unimpeachable honesty and honor; and in his death the Engineering Profession has lost a man of great worth; the community, a patriotic and loyal citizen; and his family, a loving husband and father. He is survived by his widow and three daughters.

Mr. Patton was elected a Member of the American Society of Civil Engineers on May 2, 1911. He was also a Past-President of the Duluth Section of the Society, and a member of the Duluth Engineers Club.

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## SAMUEL ROCKWELL, M. Am. Soc. C. E.*

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Died November 21, 1923.

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Samuel Rockwell, the son of William Rockwell and Susan Lawrence (Prince) Rockwell, was born in Brooklyn, N. Y., on February 20, 1847. He was of English and Colonial ancestry, a descendant of William Rockwell who settled at Dorchester, Mass., in 1630, and also, of William Brewster who came to America in 1620 in the *Mayflower* and settled at Plymouth, Mass. His father, who died when the boy was nine years old, was a Judge of the Supreme Court of the Second Judicial District of New York State and prominent in public life.

Samuel Rockwell received his preliminary education in Brooklyn and Bridgehampton, N. Y., after which he spent six years at sea, serving as mate on several vessels and receiving his Master's papers. He then entered the Sheffield Scientific School of Yale University and was graduated in the Class of 1873 with the degree of Civil Engineer.

During his college vacations, Mr. Rockwell was employed as Flagman with a party on the location of the Adirondack Railroad (now part of the Delaware and Hudson Railroad); as Levelman on the St. Paul and Pacific Railroad (now part of the Great Northern Railroad); and as Engineer on the location of the Green Bay and Lake Pipin Railroad (now part of the Green Bay and Western Railroad).

After graduation, Mr. Rockwell became Resident Engineer for the Delaware, Lackawanna, and Western Railroad Company on its improvements at Hoboken, N. J. This work included several bridges, including one over the Hackensack River, and the Bergen Hill Tunnel.

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* Memoir prepared by Willard Beahan, M. Am. Soc. C. E., and G. C. Cleveland, Esq., Cleveland, Ohio.

On the completion of this work in 1877, Mr. Rockwell entered into a partnership with Mr. Edmund Saxton, with offices at Kansas City, Mo. This firm was engaged in general contracting work and the construction of municipal water-works at St. Louis, Mo., New Castle, Ind., St. Paul, Minn., and Kansas City, Mo., until 1885.

From 1885 to 1887, Mr. Rockwell was employed as Location and Construction Engineer on the St. Paul, Minneapolis and Manitoba Railway (now a part of the Great Northern Railroad). He afterward served as Chief Engineer on the construction of the Eastern Minnesota Railway, including the tunnel at Duluth and West Superior, Minn.

In 1890, he was appointed Chief Engineer of a railroad (known as the Santa Fé, California, Extension) commencing at the then terminus of the Atlantic and Pacific Railway, Mojave, Calif., which, it was contemplated, should extend over the mountain through Bakersfield to San Francisco, and form a western outlet for the Atchison, Topeka and Santa Fé Railway. The death of the promoter and the financial situation arising from the failure of the Baring Brothers compelled the cessation of the work.

In 1891, Mr. Rockwell was engaged as Chief Engineer of the Duluth and Winnipeg Railroad, which position he resigned to enter the service of the Lake Shore and Michigan Southern Railroad Company. He remained with the latter Company until his retirement in 1918, having served as Engineer in Charge of the Michigan Southern Division from 1891 to 1899, with headquarters at Toledo, Ohio; Principal Assistant Engineer from 1899 to 1904, at Cleveland, Ohio; Assistant Chief Engineer in 1904 and 1905; and Chief Engineer from 1905 to 1912. On September 1, 1912, he was retired on account of age limit and made Consulting Engineer, in which capacity he served until 1918.

From 1918 to his death on November 21, 1923, Mr. Rockwell was engaged in private practice at Cleveland, Ohio, as a Consulting Engineer.

He was married on June 7, 1881, to Miss Cordelia A. Geiger of St. Joseph, Mo., who died in 1908. Mr. Rockwell subsequently was married to Miss Ida Estelle Horton, who, with four sons, a brother, and a sister, survives him.

Mr. Rockwell never held or sought public office, choosing to be known solely by his professional work. He was of a generous disposition and disposed to be just in all his decisions. He held the good opinion of his employees for his firmness and for his kindness of heart. His early life at sea moulded his manner to a degree, and he was precipitous in arriving at conclusions and somewhat brusque in his manner of expressing an opinion or giving an order. He was always ready, however, to admit a mistake and to make prompt amends for it. Mr. Rockwell belonged to those pioneer American railroad engineers who should be honored for their courageous earnest work in the winning of the West and giving to the country its needed transportation.

He was a member of the American Railway Engineering Association, a Royal Arch Mason, and a member of the Cleveland Athletic Club.

Mr. Rockwell was elected a Member of the American Society of Civil Engineers on January 7, 1880.

## JAMES ROBINSON SCOTT, JR., M. Am. Soc. C. E.*

DIED MAY 3, 1923.

James Robinson Scott, Jr., the son of James Robinson Scott and Lou Emma Scott, was born in Champaign, Ill., on October 25, 1885. His father, who was of Scotch ancestry, and his mother, a lineal descendant of Daniel Boone, came from Shelby County, Kentucky.

Mr. Scott obtained his early education in the public schools of Champaign. He entered the University of Illinois in September, 1903, and was graduated in June, 1907, with the degree of Bachelor of Science in Civil Engineering. A good record of scholarship, combined with a quiet and unassuming manner, caused him to be well liked by both his classmates and instructors. He was a member of the Beta Theta Pi fraternity and of the Civil Engineers' Club.

After graduation, Mr. Scott was employed by the Illinois Central Railroad Company in the capacity of Designing Draftsman in the Bridge and Building Department, his attention being confined to masonry and bridge structures. The most notable work in which he was engaged at this time was the design of the draw-span of a bridge over the Cumberland River. In July, 1910, he was transferred to the field as Masonry Inspector on a street subway at Champaign. On January 1, 1911, he was placed in charge of the construction of concrete piers of the Big Muddy River Bridge near Murphysboro, Ill., a structure which consisted of a main span of 177 ft., with approach spans of 65 ft. each. While this work was under way, Mr. Scott also had charge of the construction of a street subway at Murphysboro. In September, 1911, he was again transferred to Champaign, where he assumed charge of the construction of the buildings in connection with a new freight and mechanical terminal, including a roundhouse, a machine-shop and a storeroom, together with the necessary sewerage and drainage systems. His work with the Railway Company was concluded at the end of 1911, when, on account of malarial attacks to which he had become subject, he moved to Colorado. Here, he was at once engaged by the Denver Tramway Company as Engineer of Bridges and Buildings, in which capacity he had charge of the construction and maintenance of a number of minor structures on its city and interurban lines.

In November, 1912, Mr. Scott was employed by Herbert S. Crocker, M. Am. Soc. C. E., Consulting Engineer of Denver, as Resident Engineer in charge of the field construction of the Fourth South Street Viaduct, in Salt Lake City, Utah. This structure is a highway viaduct over the yards of the Denver and Rio Grande Western Railroad System, the superstructure being of steel over the railroad tracks, with reinforced concrete approaches. Its total length is 1 845 ft. and its width 44 ft. over all.

Early in 1914, the Salt Lake City Viaduct having been practically completed, Mr. Scott was transferred to the office in Denver, where he was engaged principally on the design of reinforced concrete spans for the Colfax-Larimer Viaduct in that city. On November 6, 1914, he was engaged by the C. S.

* Memoir prepared by H. S. Crocker, M. Am. Soc. C. E.

Lambie Company, Engineers and Contractors, of Denver, as Chief Engineer, which position he held for more than two years. During this time, he had engineering charge of that Company's contract work on the Colfax-Larimer Viaduct, which consisted of the design and construction of steel forms, superintendence of the work, and the establishment of a large plant for excavating and washing sand and gravel aggregate. He later designed an extensive concrete sheep pen, built by his Company for the Denver Union Stock Yards.

In April, 1917, Mr. Scott became associated with the Colorado Builders Supply Company, of Denver, as Chief Engineer and Contracting Engineer, having charge of building designs, quantity surveys, and the sale of building materials to owners and contractors. This engagement, particularly, increased his acquaintance among business and professional men and was the means by which he gained many strong friends who respected him for his affability, his conscientious work, and his uprightness of character. He remained with this Company until a sudden complication of ailments proved too strong for his never robust health. Mr. Scott died in Denver on May 3, 1923, and was laid to rest at his place of birth.

An engineer of ability, Mr. Scott combined with his technical attainments an open mind, executive qualities, and a winning personality that would have carried him far in his chosen profession. He leaves a most honorable record of performance. His more intimate friends will not forget his modest ways, his delicate sense of humor, and those rare qualities of mind and heart that so endeared him to all with whom he came in contact. In his profession, in social life, and in sports, his courtesy and kindly consideration were never failing.

He was a member of the First Presbyterian Church of Champaign, Ill.; of Arapahoe Lodge No. 130 A. F. and A. M., of Denver; and of Colorado Consistory No. 1 (Thirty-second Degree), of Denver. He was also a member of the Western Society of Engineers, the Colorado Society of Engineers, the Denver Athletic Club, the Lakewood Country Club, and the Denver Motor Club.

Mr. Scott was elected a Junior of the American Society of Civil Engineers on March 1, 1910; an Associate Member on September 3, 1912; and a Member on April 21, 1920.

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## BENJAMIN FRANKLIN THOMAS, M. Am. Soc. C. E.*

Died April 14, 1923.

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Benjamin Franklin Thomas, the eldest son of John N. and Hannah Hull Thomas, was born near Ironton, Ohio, on May 23, 1853. His primary education was obtained in the schools of Ironton. He finished his collegiate education, having been graduated with the degree of Civil Engineer, at the National Normal University, Lebanon, Ohio.

* Memoir prepared by William M. Hall and H. C. Corns, Members, Am. Soc. C. E.

In 1871, Mr. Thomas commenced his professional engineering career as Assistant to the County Engineer of Lawrence County, Ohio. That work was followed by an engagement in Philadelphia, Pa., from October, 1874, to 1877, the period of preparation for and holding of the great National Centennial Exposition in that city in 1876. His principal service during that engagement was as Draftsman, thereby perfecting himself in that art, so important to an engineer. From the latter part of 1877 to early in 1879, he was engaged on land surveys, maps, and reports thereof, in the States of Louisiana and Texas. Immediately following, from 1879 to the summer of 1883, he served on the construction of the Chattaroi Railway from Ashland, Ky., up the Big Sandy River, to the Peach Orchard Coal Mines, first as an Assistant Engineer and during the last two years as Chief Engineer. That line is now a part of the Chesapeake and Ohio Railway System.

In August, 1883, Mr. Thomas was appointed U. S. Assistant Engineer in charge of the improvement of the Big Sandy River, which was one of his duties at the time of his death. From that date his service was continuous, except for a short period as Engineer for the Building Contractor on the construction of Lock No. 37, Ohio River, and, during the period of the World War, as District Engineer of the Second Cincinnati District, thereby making his term of service for the United States only 10 days short of 39 years.

During these years, one of his principal works and that which gave him greatest distinction was the improvement of the Big Sandy River with three locks and movable dams. The needle-dam was the type chosen, the first movable dams of that type built in the United States. His study and reading in relation thereto of European movable dams resulted in his paper entitled, "Movable Dams",* for which he was awarded the Norman Medal by the Society.

His paper on movable dams was soon followed by a book, the "Improvement of Rivers", which was published in conjunction with Mr. David Watt. This book has since been revised by them and published in two volumes. Since its first edition, it has had wide usage as a work of reference for those engineers employed on the design or construction of works of river improvement. Besides these works, Mr. Thomas contributed discussions on at least four papers published by the Society on river improvement, the subjects including artificial waterways;† improvement of rivers;‡ levees of the Mississippi River;§ and locks and movable dams on the Ohio River.‖

Mr. Thomas was thorough, and when required to design or construct, he devoted himself to the examination of all similar works for which he could find plans or written records. In addition, he developed a genius for mechanical invention above the ordinary. In his published writings, as a correspondent, and in official reports, he was always interesting on any subject he introduced. His great love for the higher classes of professional work of

* *Transactions*, Am. Soc. C. E., Vol. XXXIX (1898), p. 431.

† *Loc. cit.*, Vol. LIV, Pt. F (1905), p. 287.

‡ *Loc. cit.*, Vol. XLIX (1902), p. 314.

§ *Loc. cit.*, Vol. LI (1903), p. 380.

‖ *Loc. cit.*, Vol. LXXXVI (1923), p. 151.

designing, planning, supervising, and studying large engineering projects, is indicated by his choosing to remain in the Government service so may years at a uniformly small compensation and declining an offer of nearly double his salary in a field of work which did not harmonize with his finer taste.

Mr. Thomas was held in high regard and esteem by all who knew him well for his unswerving integrity and as an able and conscientious worker with an unusual degree of energy which never weakened even to the time he was carried from his office a few days before his death. No one doubts that the Government was fortunate in having such a man to serve it and assist in designing and directing its work of river improvements for such a long series of years. On hearing of his death, the Division Engineer, Col. Charles W. Kutz, Corps of Engineers, U. S. A., M. Am. Soc. C. E., wrote Mrs. Thomas, in part:

"Notwithstanding that he had almost reached the retiring age, we still felt that we needed him as a counselor and adviser, and it may be of interest to you to know that, in my capacity as Division Engineer, I recommended his continuance in service."

For twenty-eight years of his service, Mr. Thomas resided at Louisa, Ky. Thereafter, as District Engineer, and as Principal Assistant Engineer for the District Engineer Office of the Second Cincinnati District, his residence was, for a greater part of the time, at Cincinnati, Ohio. Before moving to Cincinnati, he was accustomed to be called to the District Office for consultation on many days or weeks of each year; but he always preferred the activities of the field office to office work on reports and correspondence emanating from another.

Although he rarely attended the meetings of the Society, Mr. Thomas highly valued its publications and his membership in it. He bequeathed his volumes of *Transactions*, together with all the other volumes on engineering subjects in his library, to the U. S. Engineer Office, at Cincinnati, Ohio.

He is survived by his wife, Mrs. Ada Rice Thomas, his daughter, Mrs. Héloise Thomas Gunnell, and his grand-daughter, Willena Gunnell.

Mr. Thomas was elected a Member of the American Society of Civil Engineers on April 6, 1887.

## EBERHARD JOHN WULFF, M. Am. Soc. C. E.*

DIED OCTOBER 23, 1923.

Eberhard John Wulff was born at Cologne, Germany, on April 23, 1868, the son of Dr. Eberhard Wulff and Matilda (Glebsettel) Wulff. He attended the public schools and was graduated from the Cologne Horticultural Gymnasium, of which his father was Director.

Shortly after his graduation, Mr. Wulff came to the United States and settled in Tarrytown, N. Y., where he continued to reside for nearly thirty years, having became a naturalized citizen of the United States on August 23, 1894. His first employment was with Ward Carpenter and Sons, of Tarrytown, which firm was engaged in the general practice of engineering in and

* Memoir prepared by H. K. Bishop, M. Am. Soc. C. E., and F. W. Mills, Assoc. M. Am. Soc. C. E.

about Westchester County. Mr. Wulff remained with this firm, serving in various engineering capacities, for about five years. In 1894, he entered the employ of the Croton Aqueduct Commission, Bureau of Street Openings, City of New York, as Computer, Draftsman, and Assistant Engineer, resigning in 1896 to engage in private practice.

Later, he organized the Wulff Engineering Company, Consulting Engineers, of Tarrytown, and became its President. This firm handled a large amount of important engineering work in Westchester and adjoining counties, including the planning and construction of water-works at Cairo and Castleton, N. Y. He acted as Consulting Engineer for the Villages of Hastings, Irvington, Ossining, Briar Cliff, and also for many towns in Westchester County. In addition to his engineering work for these municipalities, Mr. Wulff was also extensively employed as a Landscape Architect by the owners of several large estates in Westchester County, and was engaged in beautifying the Sleepy Hollow Cemetery.

In addition to the private practice in which the Wulff Engineering Company was engaged, Mr. Wulff served for almost nine years as County Engineer for Westchester County, at the same time acting as Village Engineer for Tarrytown. During this period, he was charged with the construction and maintenance of many miles of important highways and bridges and in co-ordinating a system of connected roads throughout the County.

In 1917, Mr. Wulff was appointed Senior Highway Engineer in the United States Bureau of Public Roads, Department of Agriculture. His first assignment in this capacity was as Engineer in Charge of the Federal Aid work in the State of West Virginia. He was transferred later to the State of Ohio, and, in 1919, was attached to the Washington Office of the Bureau. After his assignment to the Washington Office, his duties consisted almost wholly in the review and criticisms of specifications submitted to the Bureau by the States for Federal approval. His long experience, carefulness, and thorough scientific training, made him particularly valuable in this line of work.

On October 16, 1902, he was married to Miss K. Louise Tanner, who died in Washington, D. C., in November, 1919. On June 7, 1922, he was married to Miss Fanny Tuley, of Washington, D. C., who survives him. His death occurred at Washington, on October 23, 1923, and he was buried in the Sleepy Hollow Cemetery, at Tarrytown.

Mr. Wulff was elected a Member of the American Society of Civil Engineers on May 6, 1914. He was also a member of the American Society for Municipal Improvements.

## ALVIN BARTHOLDI FOX, Assoc. M. Am. Soc. C. E.*

DIED DECEMBER 8, 1923.

Alvin Bartholdi Fox, the son of the late Frederick and Maria L. (Bohr) Fox, was born on October 29, 1886, at Perth Amboy, N. J.

* Memoir compiled from information on file at the Headquarters of the Society.

He received his education in the public schools of Perth Amboy and at Rutgers College, from which he was graduated in June, 1908, with the degree of Bachelor of Science. In 1912, he received the degree of Civil Engineer.

In September, 1908, he entered the employ of Morgan F. Larson, Assoc. M. Am. Soc. C. E., then County Engineer, first, as Rodman and, subsequently, as an Assistant. From November, 1909, to March, 1910, Mr. Fox served as Assistant to the Superintendent of Construction of the Perth Amboy Plant of the American Smelting and Refining Company, and, later, he was made Draftsman in the Construction Department of the Company.

In June, 1910, he was taken into partnership by Mr. Larson, under the firm name of Larson and Fox, Engineers and Surveyors, which partnership continued until his death. In 1912, he was appointed County Engineer of Middlesex County, New Jersey, and also Resident Engineer for the State Highway Department of New Jersey, which appointment he held for one year.

In 1918, Mr. Fox assisted in the organization of the Union Garage Company of Perth Amboy, and since that date he had been Secretary, Treasurer, and General Manager of the Company, the success of which was largely due to his efforts. At the time of his sudden death, he was engaged in the construction of the new building being erected by the Company.

On July 15, 1912, Mr. Fox was married to Miss Marjorie M. Brown, of New Oxford, Pa., who, with three children and his mother, survives him.

Mr. Fox was a member of the Perth Amboy Democratic Club. He was also a member of the Perth Amboy Automobile Dealers' Association and of the East Jersey and Raritan Yacht Clubs, and Treasurer of the County Engineers' Association of the State of New Jersey.

Mr. Fox was elected an Associate Member of the American Society of Civil Engineers on May 15, 1917.

## HARRY WILLEY KENNEY, Assoc. M. Am. Soc. C. E.*

Died March 8, 1923.

Harry Willey Kenney, the son of Wentworth B. and Mayme M. Kenney, was born in Lynn, Mass., on November 27, 1890.

After receiving his preliminary education, he entered Brown University, from which he was graduated in 1912, with the degree of Bachelor of Science in Civil Engineering. At odd times, while at the University, Mr. Kenney was employed by City Engineer's Office, Providence, R. I., chiefly on record maps of the sewer system. During his Senior year, he was engaged as Assistant to the City Engineer of Cranston, R. I., mainly on street improvements.

From June, 1912, to November, 1915, he served as Draftsman with the American Bridge Company, at Pencoyd, Pa., and Brooklyn, N. Y., on the detailing and fabrication of commercial steel structures, and from November, 1915, to May, 1917, he was engaged as Design Draftsman with the Baltimore

* Memoir prepared by C. B. Sadler, Assoc. M. Am. Soc. C. E.

and Ohio Railroad, at Baltimore, Md., chiefly on railroad and highway bridges of steel and reinforced concrete.

On the entry of the United States into the World War, Mr. Kenney offered his services to the Navy Department and was assigned to the Yard Development Section of the Bureau of Yards and Docks. Here, as Chief of Squad, he had charge of the preparation of the contract plans of many large steel structures for Navy Yards and Stations on both the Atlantic and Pacific Coasts.

In April, 1921, he was commissioned a Lieutenant (Junior Grade) in the Corps of Civil Engineers, Public Works Department, U. S. Navy. After brief assignments at the Naval Academy, Annapolis, Md., and the Naval Operating Base, Hampton Roads, Va., Mr. Kenney was transferred to the Eleventh Naval District, San Diego, Calif., where he remained until his death.

The principal projects completed under his direction in San Diego were the Fleet Supply Storehouse, a six-story building of reinforced concrete, and the first thirteen buildings at the Naval Training Station at Loma Portal.

He is survived by his widow, who was Miss Mary L. Craumer, of Baltimore, Md., and two children, Dorothy C. and Harry W. Kenney, Jr.

Mr. Kenney was elected an Associate Member of the American Society of Civil Engineers on January 19, 1920.

## RICHARD BARCLAY REASONER, Assoc. M. Am. Soc. C. E.*

### Died July 9, 1923.

Richard Barclay Reasoner was born on September 16, 1887, at Morrisonville, Ill. After receiving his public school education, he entered the Department of Civil Engineering, University of Illinois, in 1904. After two years at the University, he began practical work as a Rodman with the Eastern Colorado Power Company. He served subsequently as Recorder, with the U. S. Geological Survey; as Chainman, with the Missouri, Kansas and Texas Railway; as Recorder, with the Mississippi River Commission; and as Instrumentman on various projects.

In 1909, Mr. Reasoner entered the service of the Oregon Short Line Railroad Company as Division Engineer on railroad construction. On the completion of this work, he was employed on various other construction projects, including the line of the Portland Railroad and Navigation Company, from Hillsboro to Tillamook, Ore.; the construction of the Sandy River Bridge, on the Mt. Hood Railway; as Locating Engineer for the Utah Railway Company; and as Contractor's Engineer with the Utah Construction Company.

In June, 1915, Mr. Reasoner again entered the employ of the Oregon Short Line Railroad Company as an Assistant Engineer, in the Maintenance of Way Department. He was promoted, in January, 1917, to Division Engineer of the Utah-Montana Division, serving in this capacity until July, 1918, when he was commissioned a First Lieutenant in the Engineers' Reserve Corps, and was immediately sent overseas with 44th Engineers. From November, 1918, to

* Memoir prepared by B. H. Prater, Engr., Maintenance of Way, Oregon Short Line Railroad, Pocatello, Idaho.

January, 1919, he served as Division Engineer of the Le Mans Division, and from January to June, 1919, as Division Engineer of the Rennes Division, Sixteenth Grand Division.

On his return to the United States in July, 1919, Mr. Reasoner resumed his former position as Division Engineer of the Oregon Short Line Railroad Company, which he retained until the time of his last illness.

As an Engineer, Mr. Reasoner was capable, industrious, and conscientious to a high degree. His earlier general experience, combined with his railroad engineering work, had given him the necessary qualifications for a successful career, and as he had found favor with the officers and employees in the railroad service, he showed every promise of going far in that work.

On October 31, 1911, Mr. Reasoner was married to Miss Alberta Speers, of Salt Lake City, Utah, who, with a daughter, Ruth Beverly, survives him.

Mr. Reasoner was elected an Associate Member of the American Society of Civil Engineers on January 19, 1920.

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## LESLIE EDWARD PIERCE, Jun. Am. Soc. C. E.*

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DIED OCTOBER 20, 1923.

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Leslie Edward Pierce, the son of William B. Pierce, was born in Silver Springs, Fla., on March 17, 1895.

He received his technical education at Cornell University, from which he was graduated in 1916 with the degree of Civil Engineer. While at Cornell, he was active in college affairs and was also interested in athletics, having served as Chairman of the College Athletic Committee during his Senior year.

Following his graduation in May, 1916, Mr. Pierce entered the employ of the J. G. White Corporation as an Assistant to the Engineer on the construction of a power-house for the Connecticut Company at New Haven, Conn., including the installation of 7 500-kw. steam turbines and 5 000-h.p. boilers, with all the necessary steam accessories. In March, 1917, he was given complete charge of this work.

From September, 1917, to February, 1918, Mr. Pierce trained as a Cadet Officer in the Aviation Service. He was then commissioned and detailed to the command of one flight of the 184th Aero Squadron during the training period, in responsible charge of the maintenance of six aeroplanes, with power plants.

In May, 1918, Mr. Pierce was assigned to the First Army School for Aeronautical Engineers at the Massachusetts Institute of Technology, where his studies included the design of aeroplanes and airships, fluid and rigid dynamics, power plants, and aeroplane material. On the completion of this work, he received a certificate from the Institute as having been graduated from the First Army College of Aeronautical Engineering.

In September, 1918, he was appointed Assistant to Lt.-Col. V. E. Clark, at McCook Field, Dayton, Ohio, on the design of aeroplanes, and from January

* Memoir compiled from information on file at the Headquarters of the Society.

until June, 1919, he was in charge of the Structures and Aerodynamics Branch of the Aeroplane Section at McCook Field, supervising the design and testing of aeroplane structures and parts, as well as aerodynamical investigations, aeroplane operation, and calculations.

In the latter part of 1919, Mr. Pierce resigned from the Air Service and accepted a position as Engineer with the Aeronautical Corporation of Paterson, N. J., which position he retained until his death at Stamford, Conn., on October 20, 1923.

Mr. Pierce was elected a Junior of the American Society of Civil Engineers on October 14, 1919.